Carbon Fibre Reinforced Polymer Strengthened Slabs Effect on Punching Shear

A Thesis Submitted to Khartoum University in Partial Fulfilment of the Requirements for the Degree of MSc in Structural Engineering

BY

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Dedication

To whom

Give me the strength

to continue . . . .

My family

ACKNOWLEDGMENT

First all praise and thanks to ALLAH, for the ability to go through until finally complete this work.

I would like to express my sincere gratitude to Dr. Ali Zaroug Mohamed, my supervisor. His technical guidance was crucial for the assimilation of the basic concepts. He taught me how the accuracy is the first demand to achieve the goal.

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ABSTRACT

This thesis presents an experimental study on the punching strengthening of reinforced concrete slabs using CFRP plates.

Six specimens with two different reinforcement ratios-two series each of three specimens- were prepared for this study. All specimens are two-way square slabs with a central column stub. For each series one specimen was left un-strengthened, the control specimen, the second specimen was strengthened with CFRP plates placed at the middle of the first expected punching shear zone in the four directions and the third specimen was strengthened with two plates- placed side by side- bonded to the slab at the face of the central column stub in the four directions. All laminates were extended for the full length of the slab to reduce the risk of deboning failure.

The failure for all the strengthened slabs was due to debonding of the laminates which led to premature failure of the slabs; without increase in the ultimate load compare with the un-strengthened slabs.

The strengthened slabs showed a remarkable improvement in the service condition-deflection and cracking- but a brittle mode of failure.

Depending on the experimental results the use of CFRP plates for punching shear strengthening for slabs requires reliable means of preventing debonding.

مستخلص

هذه الدراسة تقدم دراسة معمليه لتقوية البلاطات الخرسانيه للاختراق باستخدام شرائح من الياف التسليح الكربونيه الملتصقة خارجيا على سطح الخرسانة.

استخدمت في هذه الدراسة ستة عينات بنسبتي تسليح أساسيتين مختلفتين- قسمت لمجموعتين كل مجموعه بها ثلاثة عينات- كل العينات كانت عبارة عن بلاطات مسطحه ذات اتجاهين، مع وجود عمود صغير في المنتصف، في كل مجموعه تركت عينه واحده من غير تقويه (عينة التحكم)، العينة الثانية قويت باستخدام شرائح من الياف الكربون البوليمريه لصقت على سطح الخرسانه فى منتصف منطقة الاختراق المتوقعه فى الاربع اتجاهات حول العمود، العينة الثالثة قويت باستخدام شريحتين من الياف الكربون البوليمريه جنبا الى جنب لصقت على سطح الخرسانة عند حافة العمود في الأربعة اتجاهات حول العمود، كل الشرائح مدت على طول البلاطة لتقليل خطر الانفصال.

كان الانهيار لكل البلاطات نتيجة لانفصال شرائح الكربون من سطح الخرسانه مما ادى الى انهيار غير ناضج بدون زيادة فى الحمل الحدى الاقصى. أظهرت البلاطات المقوية تحسنا ملحوظا في حالة الاستخدام، حيث قلت التشققات وقيم الترخيم، لكن كان الانهيار هشا.

بناء على النتائج المتحصل عليها من التجربة، استخدام شرائح الكربون الملتصقه خارجيا لتقوية البلاطات للاختراق يحتاج الى وسائل ملائمه لمنع انفصال الشرائح من سطح الخرسانه.

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CHAPTER 1
INTRODUCTION

1.1 General:

Throughout the world, an increasing number of reinforced concrete (RC) structures are being assessed as unsafe. The reasons for this include loads greater than the design capacity arising from alteration, new stringent design codes requirements, especially for earthquakes resistance and, in some cases, deterioration of the structural members. Such structures must be strengthened or retrofitted in order to serving their intended purpose.

Recently, after the collapse of World Trade Center (WTC), and appearance of the terror phenomena, the first tendency to strengthen important existing buildings to resist explosion became dominant in the western world [53].

In general, concrete structures need strengthening for the following reasons:

- To increase live-load capacity for buildings or bridges to meet new use requirements.
- To add reinforcement to a member that has been under designed or wrongly constructed.
- To improve seismic resistance, by improving the member behavior, or improving continuity between members.
- To replace or supplement reinforcement lost by impact or corrosion.
- To improve the explosion resistance.

1.2 Strengthening Solutions:

Strengthening solutions considered in a feasibility study can range from repair of a damaged structure in order to restore its original strength to adding elements to increase its capacity. All solutions are, to a greater or lesser extent, project-specific but some general approaches are commonly used.

Repair typically involves crack injection or breaking out damaged areas and reinstating with cementation repair mortars or flowing concrete in order to restore the original strength of a structure.

In case the structural capacity is not adequate, there are various forms of strengthening techniques which might be applied to increase the capacity of the concrete structure. The most common techniques are as flows:

1.2.1 Increasing the reinforced concrete cross-section

This solution is usually readily accepted by approved authorities and owners of structures as it has a proven track record. However, loading restrictions are required while the concrete cures to an acceptable strength.

1.2.2 Adding pre-stressing to relieve dead load

This technique has also a proven track record and gains ready acceptance but loading restrictions may be required during installation and the existing structure must be capable of withstanding high local pre-stressing forces.

1.2.3 Use of plate bonding to enhance flexural capacity of beams

Steel plate bonding has been widely used and can be considered to have a proven track record. Disadvantages of the technique are the difficulty of handling the plates, the difficulty of cutting to shape, the difficulty of anchoring the plates to the concrete section without causing damage to embedded reinforcement and the need to apply and maintain corrosion protection.

1.2.4 Confinement of the concrete in compression members

This can be achieved by installing in situ reinforced concrete or prefabricated steel collars. The technique tends to be readily accepted as the increase in the cross-section can be clearly seen but with in situ reinforced concrete collars loading restrictions on the structure are required while the concrete gains strength.

Traditional retrofitting techniques do not always offer the most appropriate solutions from the practical and economical point of view.

Search for other solutions led to the manufacture of the fiber reinforced polymer (FRP) materials, these found wide acceptance and attractiveness as externally bonded reinforcement techniques using epoxy adhesive. The development of high strength-to-weight ratio, ease of fabrication and bonding and excellent resistance to electrochemical corrosion of fibre reinforced polymer (FRP) composites have given this technique even more acceptance worldwide.

1.3 Problem Statement

Reinforced concrete flat plates are commonly used structural systems. Slabs supported directly on columns provide architectural flexibility, reduced building height and give clear space due to the absence of beams. One of the most important phenomena related to flat plates is the vulnerability of these systems to punching shear failure; figure (1.1) shows partial collapse of flat slabs systems due to punching shear failure.

In order to increase punching shear strength of slab–column connections, various forms of shear reinforcement, column capitals and drop panels were investigated. Shear reinforcement in the form of bent bars, shear studs and heads were used and proved to be effective in increasing punching shear capacity.

The main theme of this research is to study the effectiveness of externally bonded carbon fibre reinforced polymer (CFRP) to enhance the shear capacity of flat slabs at the slab-column connection.

1.4 Research Scope and Objectives

The main objectives of this research are

- To provide experimental evidence for the feasibility of the carbon fibre reinforced polymer (CFRP) strengthening technique to enhance the punching shear capacity for flat slabs systems.
- Verification for the analytically based expressions proposed to evaluate the punching shear capacity of slabs and footings.

1.5 Fibre Reinforced Polymers Types and Properties:

1.5.1 Types of Fibers

The most suitable fibers for strengthening applications are glass, carbon or Aramid. Typical values for properties of different fibres are given in Table 1.1 [53]. It should be noted that , these values are for the fibres alone, not for fibre composites. The strength and modulus for manufactured composites will be lower than the values in Table 1.1[53] .They should only be taken as indicative, where necessary and actual values should be obtained from the manufacturers. All fibers have a linear elastic response up to ultimate load, with no significant yielding.

1.5.2 Properties of Fibres

The important properties of fibers are:

a. Chemical Resistant:

Carbon and Aramid fibers are resistant to most forms of chemical attack. Many types of glass fiber are attacked by alkalis but not by acids. Aramids absorb much more water than either of the other two types, which can cause problems with the resin/fiber interface.

b. Resistance to Ultraviolet Light:

Glass and carbon fibers are not affected by ultraviolet light, but Aramid fibers change color and lose some strength under ultraviolet light so they must be embedded in a resin matrix to protect them. Direct sunlight can embrittel all resins so protective paint is normally recommended.

c. Electrical Conductivity:

Aramid and glass fibers are non-conducting and hence are suitable for use close to power lines, and communications facilities. But Carbon fibers do conduct electricity, so they must be electrically isolated from any power lines; and in general the resin will be sufficient for this.

Table 1.1 Typical properties of different fibrers [53]

FIBRES	Tensile strength (N / mm^2)	Modulus of elasticity (KN / mm^2)	Elongation %	Specific density
Carbon high strength	4300 _ 4900	230 _ 240	1.9 _ 2.1	1.8
Carbon high modulus	2740 _ 5490	294 _ 329	0.7 _ 1.9	1.78 _ 1.81
Carbon ultra high modulus	2600 _ 4020	540 _ 640	0.4 _ 0.8	1.91 _ 2.12
Aramid high strength and high modulus	3200 _ 3600	124 _ 130	2.4	1.44
Glass	2400 _ 3500	70 – 85	3.5 _ 4.7	2.6

d. Compressive Strength:

The compressive strengths of carbon and glass fibers are close to their tensile strengths while that of Aramid is significantly lower.

e. Stiffness:

The elastic modulus of carbon fiber is similar to, or significantly greater than, that of steel. The Stiffness of Aramid is lower and that of glass significantly lower.

f. Impact Resistance:

Performance of fibers during impact is highly dependent on the elastic strain energy generated and absorbed. Fibers combining high strength with high elongation (tensile strength greater than 3,500 N/mm^2 and elongation greater than 2%) are most suitable for applications where impact resistance is important. Selected grades of carbon, Aramid and glass fiber can meet these requirements.

g. Fire Resistance:

Glass fibers retain strength up to their melting point (over 1000°C) while carbon fibers oxidize in air at about 650°C. Aramid fibers are not normally used above 200°C. None of the fibers will support combustion. In composites, the resin behavior will dominate the performance; most of them generate toxic smoke.

h. Health and Safety:

All fibers present negligible risk to human health in normal use. However, care must be taken when cutting and machining all composites, because fine fiber particles may irritate skin, eyes and mucous membranes, so suitable protective clothing should be worn .In addition, care must be taken when handling resins.

i. Environmental Aspect:

Aramid, glass and carbon fibers are all non-toxic and inert, and are not considered to be hazardous as waste. But for incineration, the matrix in composites may present a problem. In additions, incineration of carbon materials may release fine electrically-conductive particles into the air.

1.5.3 Fabrics

Fabrics are available in two basic forms:

a. Sheet material, with unidirectional, bi-axial and tri-axial arrangements on a removable backing sheet or woven roving.

b. Fibers pre-impregnated with resin (prepreg material), which is cured once in place, by the application of heat or by other means.

Parallel layers give unidirectional properties while a woven fabric has two-dimensional properties. In woven fabrics, about 70% of the fibres are in the strong direction and 30% in the transverse direction [53]. It should be noted that, the kinking of the fibres in the woven material significantly reduces the strength. The thickness of the material may be as low as 0.1 mm (with the fibres fixed to a removable backing sheet) and is available in widths of 500 mm or more.

1.5.4 Plates

Unidirectional plates are usually formed by the pultrusion process [53].The process enables a high proportion of fibers (generally about 65%) to be incorporated in the cross-section. Hence, in the longitudinal direction, relatively high strength and stiffness are achieved; approximately 65% of the relevant figures in Table 1.1 and the transverse strength will be very low. Plates formed by pultrusion are 1to 2 mm thick and with a variety of widths, between 50 and 100 mm with very long lengths available. Thinner material is provided in the form of coils, with a diameter of about one meter Figure (1.2). It can be easily cut to length on site using a simple guillotine.

Plates can also be produced using the prepreg process, typically plates have a fiber volume fraction of 55% and can incorporate 10% fibers (usually glass aligned at an angle of 45 ^{0} to the longitudinal axis) to improve the handling strength. Lengths up to 12 m with widths up to 1.25 m and thicknesses up to 3 mm can be produced.

Figure (1.2): Coil of carbon FRP plate [53]

1.5.5 Preformed Shells:

A preformed shell is produced by filament winding. Resin-impregnated fibers are wound round a mandrel, in the pattern required to give the required hoop and longitudinal properties. Once fully cured, the cylindrical shell is removed from the mandrel and cut longitudinally so that it can be bonded round the column as per Figure (1.3) [53] . Alternatively, shells can be formed, by hand lay-up or other processes, on the inside or outside of a suitable mould. A resin rich outer skin is normally provided to improve the resistance to sunlight and salt water. The strength and stiffness of the shell in the hoop and vertical directions depends on the type and proportion of fibers in the cross-section and the method of manufacture of the composite. They are significantly lower than the values in Table 1.1.

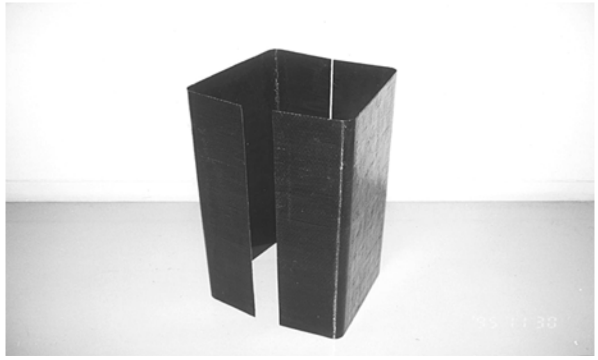


Figure (1.3): Performed CFRP plates [53]

1.5.6 Specials

Plates formed into an ' L' shape may be used as an external link to provide shear reinforcement on beams, with the lower leg of the 'L' providing the anchorage for the vertical portion. The same type of unit could be used to provide anchorage at the top of a beam, at the interface with the slab or at beam-column connections. There are various applications for this type.

1.5.7 Adhesives

The adhesives most commonly used with concrete are epoxies (usually solvent-free, two pack materials which cure at ambient temperature).The adhesives that are sometimes considered as alternatives to epoxies have certain drawbacks, such as :

- Polyester adhesives have high curing shrinkage, high coefficient of thermal expansion, subject to alkaline hydrolysis and are difficult to bond to when hardened.
- Vinylester adhesives are subject to curing shrinkage and the bond is badly affected by moisture.
- Polyurethane adhesives have high curing shrinkage, can be affected by moisture and are difficult to bond to.

The selection of the type of epoxy to be used in a particular application is governed by various factors, including the environment and the required speed of fabrication; generally the adhesive should be able to withstand a temperature not less than 50^0C in service and a glass transition temperature (Tg) between 50 and 65°C.

1.5.8 Installation Process

All installations should comply with the requirements of the Health and Safety at Work Act and the Construction Regulation. In addition, all materials must be used in accordance with the manufacturer's requirements and they must be applied by experienced personnel. It is vitally important that the manufacturer's recommendations are followed throughout and Quality assurance procedures. Each stage must be approved before starting the next stage.

1.5.9 Advantages and Disadvantages of FRP

Fiber composite strengthening materials have higher ultimate strength and lower density. The lower weight makes handling and installation significantly easier and no need for the supporting system while the resin hardens. No bolts are required to fix the fibers composites to the slab, so there is no risk of damaging the existing reinforcement.

Fiber composite materials are available in very long lengths, which with the flexibility of the material will simplify installation. Laps and joints are not required as the material can take up irregularities in the shape of the concrete surface. The material can follow a curved profile and can be readily installed behind existing services. Overlapping is only required when strengthening in two directions, but this is not a problem since the material is very thin. The materials - fibers and resins - are durable if correctly specified, and require little maintenance.

The use of fibre composites does not significantly increase the weight of the structure or the dimensions of the members. The latter may be particularly important for bridges and other structures with limited headroom such as tunnels. In terms of environmental impact and sustainability, studies have shown that the energy required to produce FRP materials is less than that for conventional materials. Because of their light weight, the cost of transport of FRP materials is minimal. These factors made strengthening process significantly simpler and quicker as compared to the conventional methods.

The main disadvantage of external strengthening of structures with fiber composite materials is the risk of fire, vandalism or accidental damage; thus protection of the strengthening will be required. A particular concern for bridges over roads is the risk of soffit reinforcement being hit by over-height vehicles. Damage to the plate strengthening material can reduce the overall factor of safety but it is unlikely to lead to collapse.

Experience of the long-term durability of fiber composites is not yet available. This may be a disadvantage for structures designed for a very long design life but this can be overcome by appropriate monitoring.

A perceived disadvantage of using FRP for strengthening is the relatively high cost of the materials. However, comparisons should be made on the basis of the complete strengthening exercise; in certain cases the costs can be less than that of steel plate bonding. A disadvantage in the eyes of many clients will be the lack of experience of the FRP techniques and suitably qualified staff to carry out the work.

1.6 Thesis Arrangement

This thesis falls into five chapters as follows:

Chapter (1): Introduction

This chapter enlists the reasons for strengthening, strengthening solutions, FRP materials types and properties with their installation process, statement of the problem , objectives of the study and thesis arrangement.

Chapter (2) : Literature Review

This chapter summary the previous studies for strengthening of different structural elements using (FRP), their ideas and findings.

Chapter (3): Design Approach for the Slab Column- Connection

In this chapter the basic principles underlying the design are presented.

Chapter (4): Experimental Specimens and the Test Procedures

The experimental set up, the experimental program and the experimental tests results are presented in this chapter.

Chapter (5) Analysis of Results and Conclusions

This chapter comprises the analysis of the results, the conclusions and the recommendations for the future work.

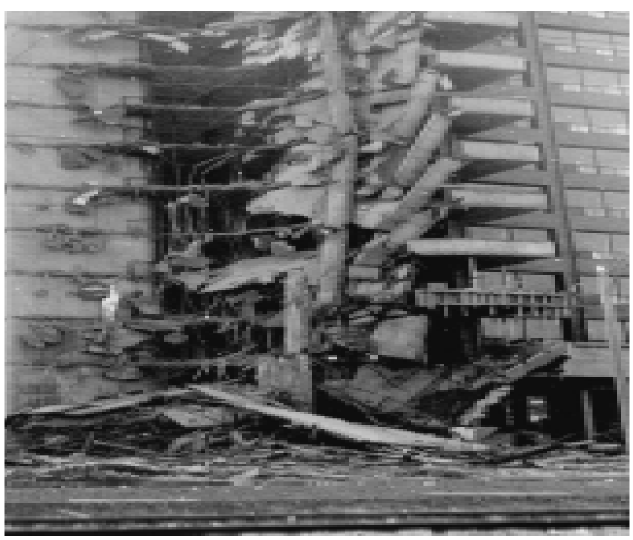


Figure (1.1): Partial collapse of a flat-plate structure due to punching shear failure [54]

CHAPTER (2)
LITERATURE REVIEW

2.1 Introduction:

The conventional strengthening methods do provide enough additional strength, however they are elaborate, difficult to install, expensive, and aesthetically not pleasing [49]. The development of FRP materials in various forms and configuration which can take the form of bars, cables, 2-D and 3-D grids, sheet materials and laminates offers an alternative strengthening solution, their light weights, high strength-to-weight ratio, ease of handling and application and corrosion resistance are some factors that are advantageous in repair, retrofitting and rehabilitation of civil engineering structures.

2.2 Conventional Strengthening Methods

2.2.1 Section Enlargement:

This method of strengthening involves placing additional bonded reinforced concrete to an existing structural member in the form of an overlay or a jacket. With section enlargement, columns, beams, slabs and walls can be enlarged to increase their load-carrying capacity or stiffness. But the use of this technique is always limited by the difficulty and, sometimes the impossibility, of the installation process [56].

2.2.2 External Post-tensioning:

This type of upgrading is generally applied where tension cracks appear. An external compressive force is applied to the structural member using post-tensioned (stressed) cables fixed to the surface of the member. Because of the negligible weight of the repair system, this technique is effective and economical, and has been employed with great success for cases of excessive deflections and cracking in beams and slabs. Members must be stiff enough to resist the external compressive force, and the anchorage force [56].

2.2.3 Span Shortening:

Span shortening system is accomplished by installing additional supports underneath existing members to reduce their spans. Such supports include structural steel members and cast-in-place reinforced concrete members. This technique can be used for slabs and beams [56].

2.2.4 Bonded Steel Elements:

In this method, steel elements (plates, channels, angles or built-up members) are glued to the concrete surface using epoxy adhesive and bolts to create a composite system. To increase the flexural resistance, the Steel elements are bonded on the tension side for slabs and beams parallel to the main reinforcement, and the bolts are used to fix them in position due to the relatively heavy weight of the steel elements. Steel elements can also be used to improve the axial capacity of columns (steel jackets) [56].

2.3 Historical Background OF FRP

The experiences in the most important countries that have best dealt with the FRP will be summarized as follows:

a. United States:

In the United States the interest in fiber based reinforcement for concrete structures started in 1930's. However, actual development and research activities into the use of FRP materials for retrofitting concrete structures started in the late 1980's [43]. FRP materials have quickly moved from the state-of-the-art to mainstream technology and their applications in many fields had started [12, 18, 19]. In addition, there is continuous research done by the state department of transportation (DOTS) for pursuing the use of FRP for repair and retrofit of transportation structures [37].

In 2002 the ACI Committee 440 developed a guide (ACI 440 2002) [1] for the design and construction of externally bonded FRP systems for strengthening concrete structures. Many of the Innovative Bridge Research and Construction (IBRC) projects have been or are being conducted that involve the bonding of FRP composites to concrete structures [30] in addition to numerous individual projects [3, 29, 36, 45].

b. Europe:

Research on the use of FRP in concrete structures began in Europe in the 1960's [44, 55, 14], but pioneering work took place in the 1980's in Switzerland and resulted in successful practical applications [30, 31]. Earlier applications of FRP strengthening in Europe were performed in 1991 on the Ibach Bridge, Switzerland; and Kattenbusch Roadway Bridge in Germany [42]. A pan-European collaborative research (EUROCRETE) was established in 1993 to develop FRP reinforcement for concrete and included partners from the United Kingdom, Switzerland, France, Norway and Netherlands. Near surface Mounted (NSM) carbon FRP strips were used to rehabilitate the "Tobel Bridge" in Southern Germany in 1999 [8] . In 2000 "Design guidance for strengthening concrete structures using fibre composite materials" was established by the UK Concrete Society [49] -Technical Report No 55. In 2001 the International Federation for Structural Concrete (FIB) Task Group 9.3 on FRP Reinforcement for Concrete Structures published a bulletin on design and guidelines for externally bonded FRP repair systems [CEB-FIP 2001] [13].

c. Japan:

Together with Europe, Japan developed the first FRP application for construction in the early 1980's [43]. A sudden increase in the use of FRP was attained after the 1995 Hoboken Nanbu earthquake. As of 1997, the Japanese led in FRP reinforcement usage, with 1000 demonstration commercial projects as well as the introduction of design provisions for FRP in the standard specifications of the Japan Society of Civil Engineers (JSCE) [22].

d. Canada:

The use of FRP for repair and strengthening of concrete structures began in earnest in the late 1980's [43]. A significant international research break through was achieved in 1998 by strengthening Taylor Bridge in Headingly, Manitoba, with CFRP cables and bars. In 1999 a trial application of CFRP sheets as a first step in upgrading the shear capacity of the Maryland Street bridge in Winnipeg, Manitoba, was conducted. The Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures (ISIS) was established in 1995 to conduct research and development on the innovative use of FRP techniques. Canada standards Association and the ISIS have published a comprehensive manual on FRP repair systems for concrete structures [CSA S806-02] [21].

Continuous researches conducted to study the behavior of the strengthened members [slabs, beams and column] using FRP materials under flexure, shear, axial force led to relatively mature knowledge. Some of the most important contribution will be highlighted below.

2.4 Strengthening Of the Slabs Using FRP

2.4.1 Flexural Strengthening

The use of FRP in different systems bonded to the tension face of the slabs increases the flexural capacity and the stiffness of the slab and reduces the deflection. The failure mode of the strengthened slab was always de-bonding.

Finite element method and the yield line theory were often used to predict the behavior of strengthened slabs.

Tan, Tumialan and Nanni [[50]

They used different CFRP systems to increase the flexural capacity of two way simply supported slabs , (strips of laminate plates with Cold cured adhesive bonding, prestressing strips of laminate plates, wet lay-up ply of Fiber laminate sheets, near surface mounted strips of laminate bars (NSM)). They found that CFRP increased the flexural strength between 63% to 145% and remarkably reduced the deflections and crack widths, especially the prestressing CFRP system. Two modes of failure were observed. Delamination occurred in the cases CFRP with cold cured adhesive and prestressing CFRP while rupture of the CFRP reinforcement was observed in the other cases of the CFRP system.

Marzouk, Ebead and Neale [28]

They used CFRP strips and GFRP laminates to strengthen two way slabs with the scheme shown in the figure (2.1) to increase the flexural capacity of the slabs. The strengthened specimens using FRP strips or laminates showed an average gain in the load capacity of about 36% over that of the reference (un-strengthened) specimens. In addition, the strengthened specimens showed a stiffer behavior than that of the reference specimens. However, a decrease in ductility and energy absorption was recorded due to the brittle nature of the FRP materials. Debonding of FRP materials was the main cause of failure. Slabs failed soon after de-bonding occurred due to exceeding flexural cracks. None of the strengthening materials experienced rupture failure.

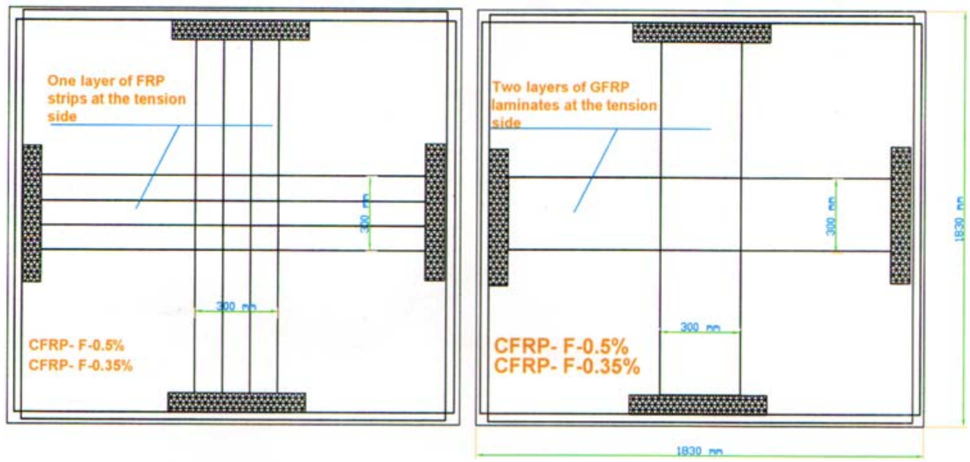


Figure (2.1): Layout of the flexural –strengthening scheme [28]

Mosallam, Mosalam [34]

They studied un-reinforced and reinforced concrete slabs repaired and retrofitted with fiber reinforced polymer (FRP) composite strips. Both carbon epoxy and glass epoxy composite systems were used in this study. They found that the FRP systems were effective in appreciably increasing the strength of the repaired slabs to approximately 500% for un- reinforced specimens and 200% for reinforced specimens. They used the finite element method and found good correlation between computational models and the experimentally determined results for both the control and the rehabilitated slabs.

Tan and Zhao [51]

They investigated one-way RC slabs with openings strengthened with externally bonded carbon fibers polymer (CFRP) systems,. They found that CFRP system proved to be effective in enhancing the load-carrying capacity and stiffness of RC slabs with openings, provided that premature failure due to CFRP debonding is excluded. They used the yield line method to analyze the strengthened slabs and found that the analytical model predicts the load carrying capacity of the strengthened slabs very well.

2.4.2 Punching Shear Strengthening

In practice, to increase the punching shear capacity of the slab, the area around the column within the zone of the critical punching shear must be strengthened. Many of the researches showed that the use of FRP materials in different pattern around the column can increase the punching shear capacity due to the increase of the flexural capacity of the slab and always the punching shear cracks start with flexural cracks. Figure (2.2) shows the widely used test scheme for punching shear for the slab [54], where the slab was supported along its four edges with the corner free to lift up and loaded through a central column stub; sometimes projecting from the tension side.

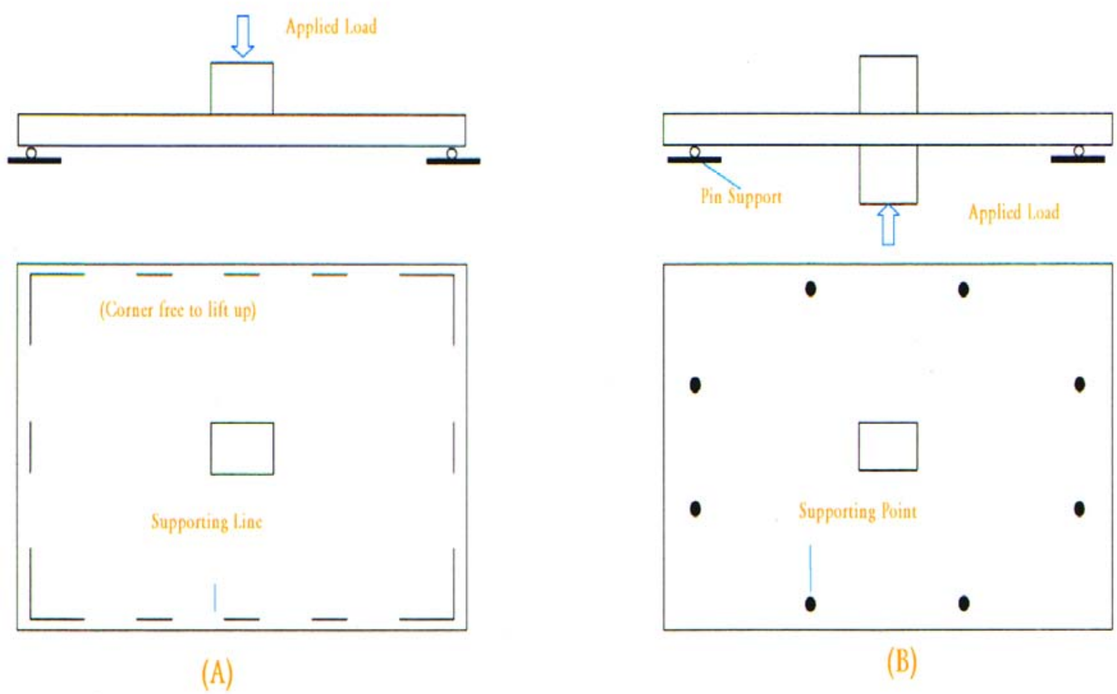


Figure (2.2): The test scheme for punching shear [54]

Erki and Heffernan [17]

Erki and Heffernan used Glass FRP (GFRP) and carbon FRP (CFRP) unidirectional sheets to strengthen two-way and one-way slabs for punching shear. They used the test scheme without the column stub projecting from the tension side of the slab. They found that the additional tensile capacity provided by the FRP sheets increased the flexural stiffness of the slabs and delayed the onset of flexural cracking to higher loads; there - by increasing the punching shear capacity.

Limam, Foret and Ehrlacher [27]

They dealt with strengthening of reinforced concrete (RC) two-way slabs with carbon fibre reinforced plastic (CFRP) strips bonded to the tensile face. The strengthened slab presented a failure mode with debonding of the external FRP strips from the slab. The strengthened slab was designed as a three-layered plate supported in four sides, which was subjected to a load in the centre. The limit analysis was used to approximate the ultimate load capacity and identify the different collapse mechanisms. Experimental results were compared with theoretical predictions.

Tan [49]

Tan used different FRP systems, namely, discrete carbon FRP plates, continuous carbon fiber sheets, and continuous glass fiber fabric to strengthening two–way slabs for punching shear. The width and spacing or the number of layers was varied. He used the test scheme for the slabs with the column stub projecting from the tension side of the slab. He found that the FRP reinforcement registered strains at failure of about 20% of their ultimate strain, and that slabs strengthened with unidirectional FRP system did not give a significant increase in punching shear resistance. However, for the slabs strengthened with bidirectional systems the punching shear strength increased according to the reinforcing index. Same reinforcing index using the CFRP sheets gave the highest punching shear strength while the CFRP plates gave the least.

Chen and Li [14]

They used glass fiber-reinforced plastics GFRPS in one and two layers in different patterns to strengthening two–way slabs for punching shear. They used the test scheme with the column stub projecting from the tension side of the slab. They found that the presence of GFRP substantially increases the punching shear capacity of slab–column connections.

Binic and Bayrakb [7]

They used closed loop of carbon fiber reinforced polymer (CFRP) in form of stirrups within the shear zone to upgrade reinforced concrete slab–column connections subjected to monotonic shear and unbalanced moment. CFRP fabrics were cut to length of 17 mm and impregnated with epoxy, then they were weaved around the holes [made in the slab by vertical tubes during the casting process] in the directions parallel to the closest edge of the loaded area in two different patterns as in Figure (2.3). They used the ACI 318-02 and yield line theory to predict the theoretical ultimate loads for the control specimens.

On the basis of the results of this study, use of CFRPS as externally installed stirrups was found to be successful in strengthening slab–column connection.

Harajli and Soudki [20]

They used CFRP sheets bonded to the tension face of the slabs in two perpendicular directions to increase the punching shear capacity of interior slab–column connections as shown in Figure (2.4). They used the test scheme with the column stub from both sides of the slab. The test results clearly demonstrated that using CFRP led to significant improvements in the flexural stiffness, flexural strength, and shear capacity of beam–column connections. The enhancement in the flexural capacity is between 26 and 73% and in the shear capacity is between 17 and 45%. The measured stress in the CFRP sheets at nominal strength varied between 22 and 69% of the ultimate tensile strength of the fibers.

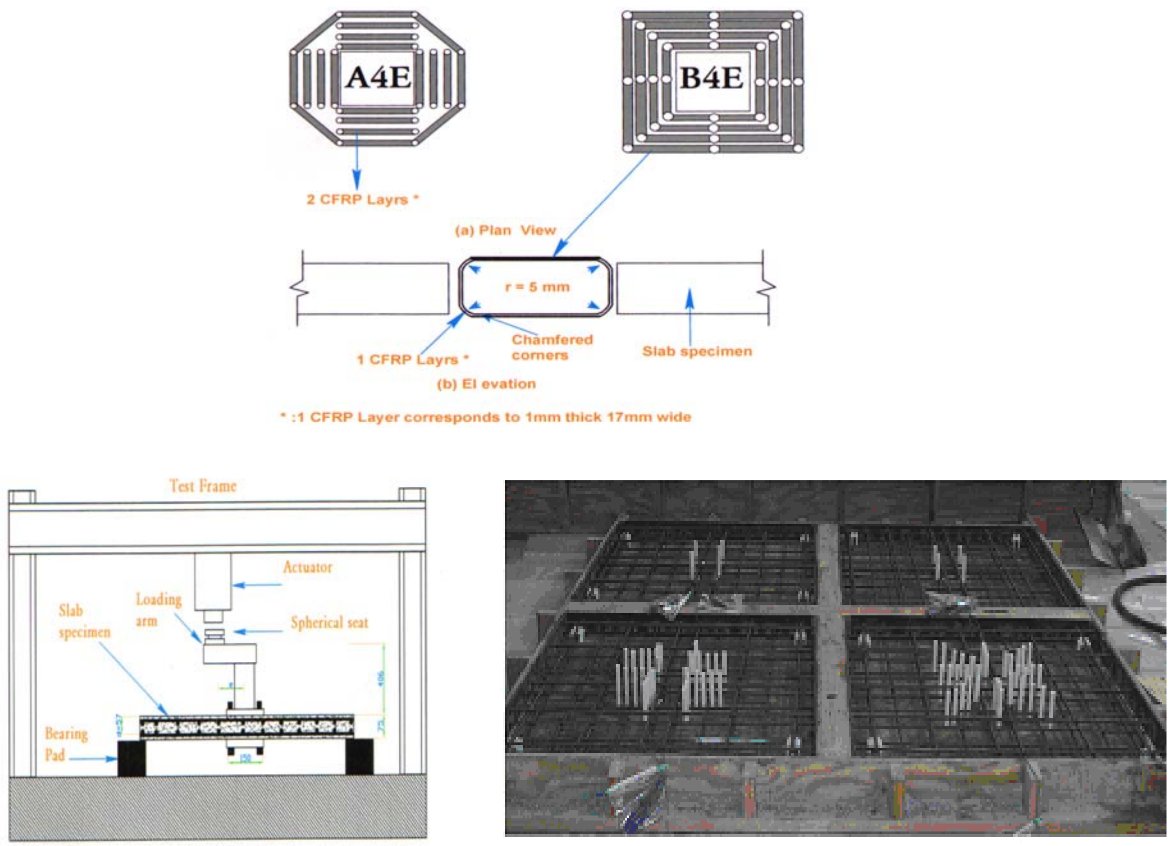


Figure (2.3): Test set up and specimen layout [7]

Soudki [46]

Soudki studied the effectiveness of FRP sheets in increasing the shear capacity of two-way slabs. He found that the use of FRP sheets in the critical negative moment region of the slab–column connection delays the formation and growth of tensile, flexural and shear cracks by increasing the flexural strength of the slab in the vicinity of the column. This consequently, improves the two-way shear resistance of the connection. He concluded that the efficiency of using FRP sheets for repair, in comparison with , an equivalent amount of reinforcing steel, is expected to increase with a decrease in the thickness of the element; thus indicating that the use of FRP sheets for upgrading slabs is promising.

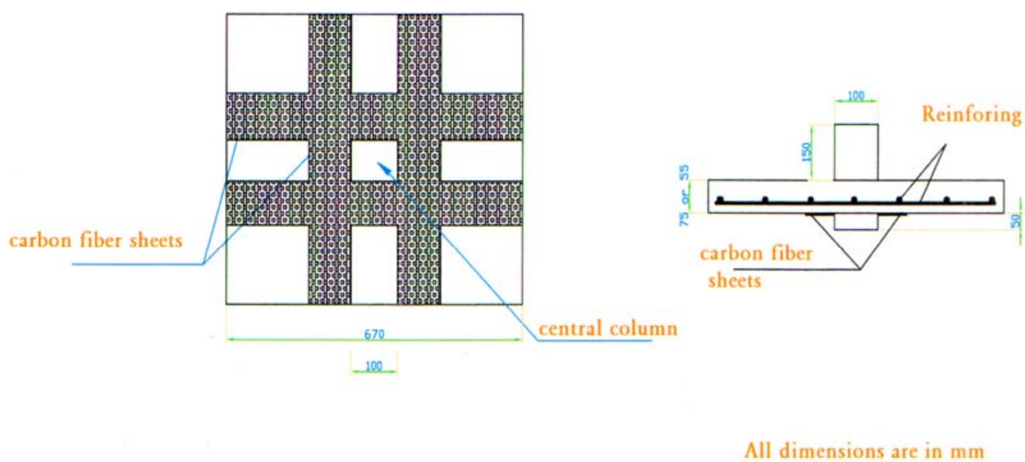


Figure (2.4): Strengthened scheme for punching shear

Soudki, Zwol and Sherping [47]

They used different systems of carbon fibre reinforced polymer (CFRP) strips to enhance the behavior of flat slab-column connections. They used orthogonal and skewed configurations of externally bonded CFRP strips, adjacent and/or offset to column face. In the orthogonal orientation, the CFRP strips were placed parallel to the internal steel reinforcement while in the skew orientation, the strips were laid in a 45-degree angle relative to the internal reinforcement. These patterns are shown in Figure (2.5). The test results clearly showed that CFRP strengthening led to an improvement in the structural behavior of slab-column connections. Depending on the configuration and orientation of CFRP strips, the increase in strength ranged between 8% and 28%.

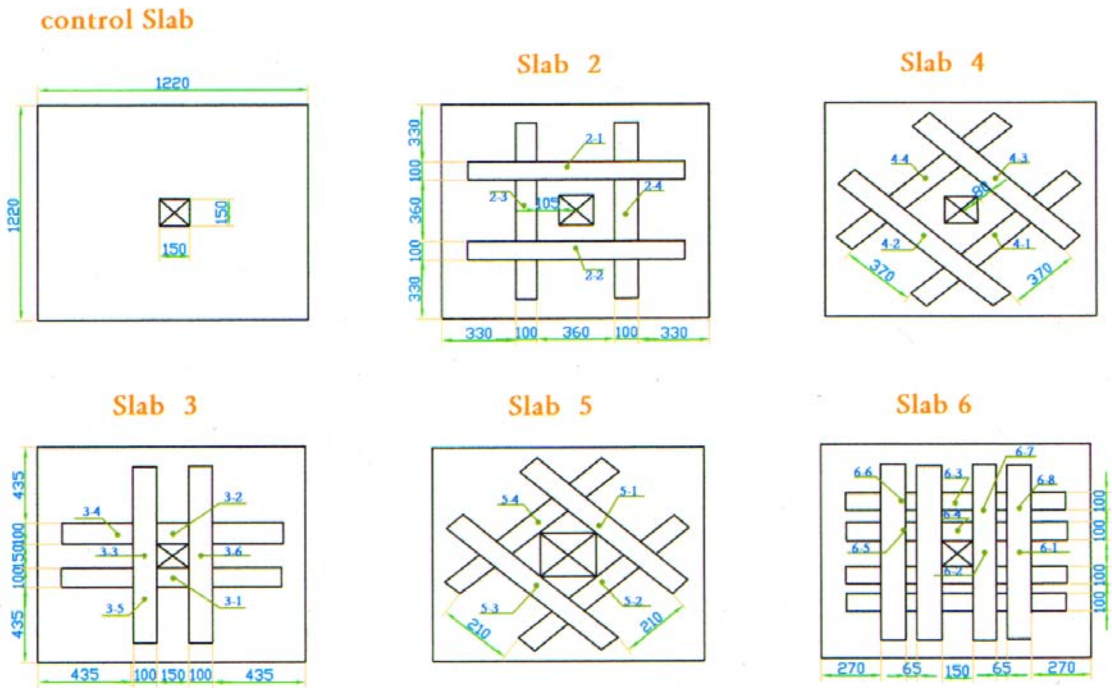


Figure (2.5): Different strengthened schemes for punching shear [47]

2.5 Strengthening of Beams Using FRP

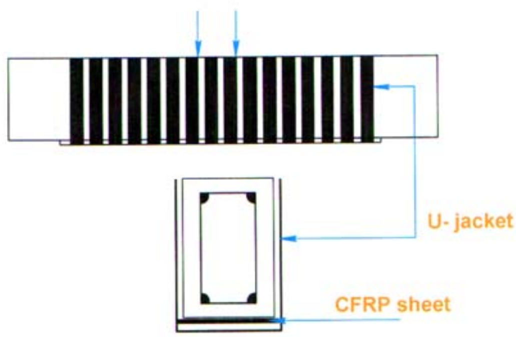
2.5.1 Flexural Strengthening of Beams:

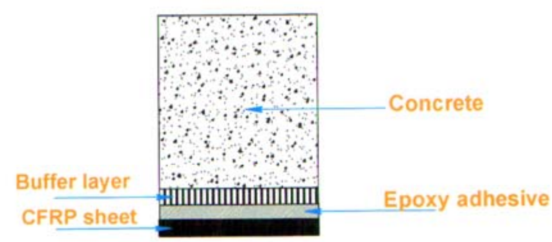
The flexural capacity of beams can be increased using FRP reinforcement bonded to the tension face of the beam parallel to the main reinforcement. The failure mode of the strengthened beams is always due to debonding of the FRP strips, especially for the large size beams. The use of a primer layer before the epoxy layer increases the strength, changes the failure mode to breakage of FRP instead of debonding and improves the ductility of the beam.

The efficiency of FRP can be greatly reduced due to premature failure caused by end peeling and shear–flexural peeling. The use of U- end wrapping system is effective in end peeling while the L- wrapping and the X –wrapping schemes can prevent both the end peeling and shear – flexural peeling.

Y.Takahashi and Y.Sato [48]

They used three different systems of CFRP to increase the flexural strength of RC beams. In the first system they only used CFRP sheets on the tension surfaces. In the second system they used a soft layer with CFRP layer on the tension face of beam and in the third system they used a soft layer and CFRP sheets on tension side plus 5-cm-wide strips of CFRP sheet wrapped around the web-as in Figure(2.6). They found that the flexural strength of the strengthened beams increased, the ductile behavior of the beam reinforced with CFRP sheet was significant and increased by using the buffer layer , U jackets , buffer layers and U jackets together . The failure mode was by breakage of the CFRP sheet.





(a) Wrapped appearance by U-jacket (b) Beam cross-section

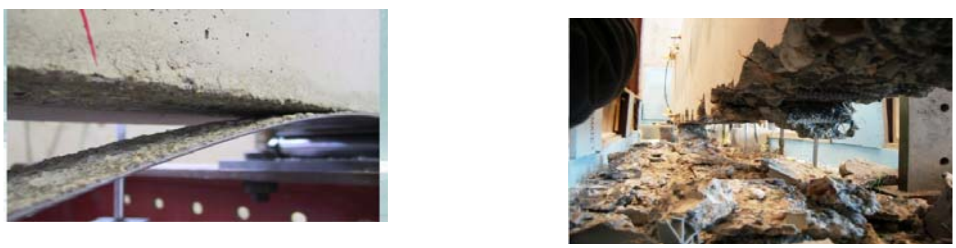
Figure (2.6): U jacket strengthened scheme with buffer layer [48]

Leong and Maalej [25]

They used different sizes of beams strengthened with CFRP sheets in different layers to increase the flexural capacity. They found that increasing the size of the beam leads to increased interfacial shear stress concentration in CFRP strengthened beams as well as reduced CFRP failure strain. But the beam size does not significantly influence the strengthening ratio, nor does it significantly affect the deflection ductility of CFRP strengthened beams.

Arduini and Nanni [4]

They described two common types of peeling failure, namely end peeling and shear-flexural peeling as shown in Figure (2.7). The end peeling results from the combination of shear and normal tensile stress localized in the vicinity of the plate end. When the principal tensile stress reaches the tensile strength of concrete, a crack initiates and propagates horizontally at the level of tension steel, ripping off the concrete cover. The shear-flexural peeling initiates at the base of flexural or shear-flexural crack, and propagates towards the support.



(a) Shear – flexural peeling (b) End peeling

Figure (2.7): Flexural peeling and end peeling failure [4]

Nurchi et al [38]

They proposed the use of multi – directional laminate at the soffit of the beam with external anchorage bolts to strengthen RC beams for flexure. They found that:

(a) The use of anchorage bolts at ends of laminates can prevent failures such as concrete rip-off.

(b) The use of bolts over the shear span significantly postponed debonding of the laminate due to vertical crack displacement.

(c) Due to the bolt anchorage, after debonding, the laminate acts as an external tension member. This results in increased the ultimate deflection and hence less brittle failure modes.

Pornpongsaroj and Pimanmas [39]

They proposed different schemes for strengthening the RC beam for flexure with end anchorage. They used U-, L- and X-wrappings anchorages. They found that without wrapping the FRP strengthened beams showed little strength improvement over the un-strengthened beam. For U-wrapped strengthened beam, end peeling could be prevented but shear-flexural peeling took place instead. For L and X-wrapped beams, no sign of detrimental peeling was observed and the beam failed in flexural concrete crushing mode.

2.5.2 Shear Strengthening of Beams:

The shear capacity of the beam can be increased by using FRP reinforcement bonded to side faces of the beam in the area of the maximum shear. The failure mode is always debonding, so to utilize the maximum efficiency of the FRP system, the FRP reinforcement must be anchored at their ends. There are different ways of anchorages such as using FRP sheets at the bottom of the beam to anchor the vertical FRP plates, or wrapping the FRP reinforcement around the flange of the beam through drilled holes.

Chen and Tengb [15]

Chen and Tengb developed a new simple, accurate and rational design proposal for the shear capacity of FRP-strengthened beams for which debonding is the failure mode. They substantiated the validity of the new proposal by the available experimental data.

Khalifa and Nanni [23]

They investigated the influence of some parameters in the capacity of the CFRP strengthening beam for shear. These parameters were CFRP amount, ply combination and CFRP end anchorage. They found that externally bonded CFRP can increase the shear capacity of the beam significantly and the most effective configuration was the U-wrap with end anchorage.

Brown and Hamilton [9]

They used different schemes of CFRP plates and sheets to strengthen corrosion damaged reinforced concrete beams for shear as in Figure (2.8). The repaired beam showed good performance and the shear capacity was increased.

Lee and Mahaidi [24]

They Found that, a maximum increase in shear capacity of 81% was achieved in T-beams strengthened with the external CFRP reinforcement. The presence of the CFRP external reinforcement did not delay the initial formation of shear cracks but impeded its propagation and growth. The deformation mechanisms in the strengthened beams were similar to those of the un-strengthened beams.

Adhikaryh, Mutsuyoshi and Ashraf [2]

They Used carbon fibre reinforced polymer (CFRP) and Ararmid fibre reinforced polymer (AFRP) uni-directional sheets with end anchorage to increase the shear capacity for RC beams. The sheets were applied only in the shear span and anchored at the top of the beam with different distances as detailed in Figure (2.9). They suggested two separate equations to calculate contribution of FRP sheet to the shear capacity of strengthened beams (v_f). First when failure is likely to occur due to sheet debonding and second when bonded anchorage of FRP sheet is provided to the beams. They found that

- Effectiveness of externally bonded CFRP and AFRP sheets for shear strengthening of RC beams was confirmed.
- A maximum of 123% increase for CFRP and 118% increase for AFRP strengthened beams in their shear capacity was achieved.
- FRP sheet with bonded anchorage is much more effective than U-wrap scheme and is an effective way to delay or evade sheet debonding.
- Bonded anchorage of sheet resulted in a decrease of interface bond stress and an increase in effective strain of FRP sheet at failure.
- The proposed equations can be used to estimate the contribution of FRP sheets (vf) to the shear capacity of RC beams with satisfactory accuracy.

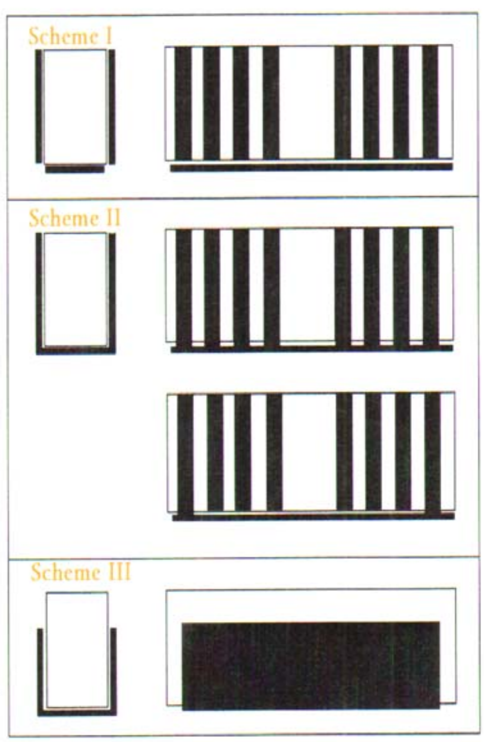


Figure (2.8): Different strengthened schemes for shear in beam [9]

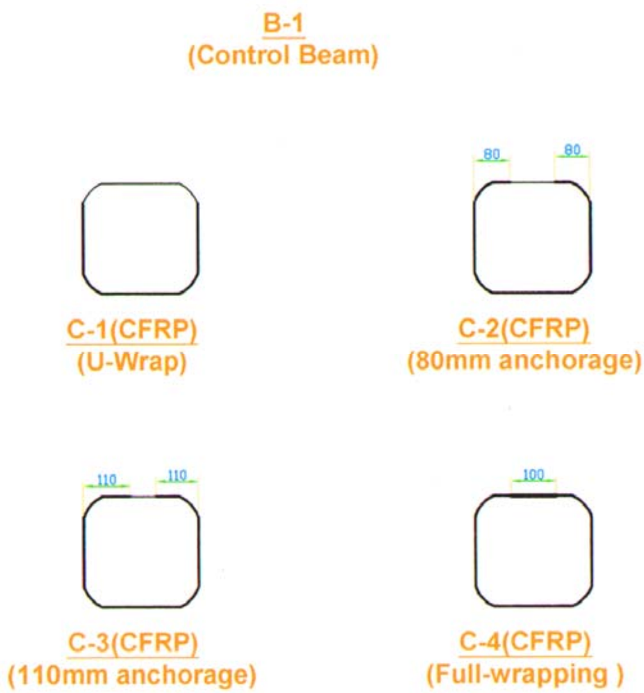


Figure (2.9): Strengthening techniques using U jacket with end anchorage [2]

Melo, Araujo and Nagato [32]

They proposed new ways of anchorage for the CFRP sheets used for shear strengthening. In the first method the CFRP sheets are enveloping only the web of the beams and anchored at the bottom of the slab with horizontal strips and in the second method the CFRP sheets are rapped around the rib of the beam and continued to the top holes drilled in the slab. They found that the second approach was more effective than the first one but it is much more laborious and messy.

2.6 Strengthening of column using (FRP):

The major parameters affecting the behavior of concrete columns confined with external FRP are the type of fibers; the number of layers, the shape of cross-section, and layout of fibers which contribute much to the behavior of confinement effectiveness when bending action is introduced.

Li, Moulsdle and Hadi [26]

They Found that the use of FRP reinforcement to confine high strength concrete column:

- Enhances the ductility of the column.
- Enhances the structural performance of concrete columns under eccentric loading to some extent. However, the enhancement is not as significant as that of columns under concentric loading. This was attributed to the fact that an eccentric loading once engaged, induces bending action together with the axial compression.
- For the circular specimens under concentric loading, the number of layers of FRP materials is one of the major parameters having a significant influence on the behavior of specimens.
- The fibre layout is one of the major factors that affect the effectiveness of confinement, especially when eccentricity is introduced.
- Taking the expensive costs involved into consideration, external confinement with Carbon fibres is not suggested for strengthening of columns under largely eccentric loading.

Tastani and Pantazopoulou [52]

They upgraded several columns by means of FRP jacketing after being conditioned to accelerate electrochemical corrosion, and were subsequently tested to failure under concentric compression; A significant increase in strength and deformation capacity was observed in all cases. Failure was rather brittle particularly in CFRP jacketed specimens.

Prota, Manfred and Cosenza [40]

They tested rectangular columns, upgraded by unidirectional Glass FRP laminates, subjected to axial load. Their results confirmed that the confinement with GFRP laminates could represent an effective technique for the strengthening of RC rectangular columns. Significant increase in both strength and ultimate axial strain was achieved.

2.7 The research problem

In this research enhancement of the shear capacity of flat slabs at the slab-column connection will be addressed. The contribution of externally bonded CFRP strips in different arrangements will be experimentally studied.

CHAPTER (3)

Design Approach for Shear strengthening using (FRP) Laminates for the slab column – connection

3.1 Introduction

There are several national guidelines for the selection of FRP systems and the design and detailing of structures incorporating FRP reinforcement. However, there exists a divergence of opinion about certain aspects of the detailing between these guidelines. This is to be expected as the material is relatively newly developed worldwide. Much research is being carried out at institutions around the world and it is expected that design criteria will continue to be enhanced as the results of this research become known in the coming years.

The design approach here will follow the guideline of the Concrete Society (UK) 2000 [53] and BS 8110 1985 where the design of [FRP] strengthening systems will be based on limit state principles. The aim of limit state design is the achievement of an acceptable probability that the structure being strengthened will perform satisfactorily during its design life.

3.2 Ultimate Limit States:

The design of FRP strengthened systems mainly must be checked for bending, shear and compression as well as checks for plate separation; a condition peculiar to FRP strengthened structures. Since structural strengthening invariably increases the stiffness of flexural members which in turn increases the risk of brittle failure, a check on ductility will also be necessary.

3.2.1 Mechanical Properties of Materials

A. Properties of Concrete and Steel Reinforcement

The design strength for the concrete and the steel reinforcement will be the characteristic strength divided by partially safety factor (γ_{mc} =1.5, γ_{ms} =1.15) for concrete and steel respectively as recommended by BS 8110 1985. Moreover, in order to avoid permanent deformations, the designer must check that the steel reinforcement at service loads does not yield. Accordingly it is recommended to increase the partial safety factors for steel reinforcement to 1.25 in performing this check [53].

B. Properties of FRP Material

All design properties of FRP materials are based on the actual properties obtained from the manufactures divided by the appropriate safety factors to account for the uncertainties associated with the material itself γ_{mf} , with its manufacturing route γ_{mm}, and with changes in material properties with time γ_{mE} .

Table 3.1and Table 3.2 show different values for γ_{mf} and γ_{mm} respectively.

(i) Design Strength of FRP (f_{fd})

The design strength will be the characteristic mechanical strength f_{fk} divided by γ_{mF}

$$f_{fd}= f_{fk} / (\gamma_{mF} * \gamma_{mE}) \quad (3.1)$$

$$\gamma_{mF} = \gamma_{mf} * \gamma_{mm} \quad (3.2)$$

Where

f_{fd} = design strength of FRP

f_{fk} = characteristic mechanical strength of FRP

γ_{mf} = partial safety factors for the FRP strength depending on the type of fibre given in Table 3.1.

γ_{mm} = partial safety factors for the FRP strength depending on the manufacturing route given in Table 3.2.

$\gamma_{\mathbf{mE}}$ = Partial safety factor for modulus of elasticity for the FRP given in Table 3.3.

Table 3.1 Recommended values of partial safety factors (γ_{mf}) [53]

Material	Partial safety Factors (γ_{mf})
Carbon FRP	1.4
Aramid FRP	1.5
Glass FRP	3.5

Table 3.2: Recommended values of partial safety factors (γ_{mm}) [53]

Type of system (method of application of manufacture)	Partial safety factor (γ_{mm})
Plates	
Pultruded	1.1
Prepreg	1.1
Preformed	1.2
Sheet or tapes	
Machine-controlled application	1.1
Vacuum infusion	1.2
Wet lay-up	1.4
Prefabricated (factory-made) shell	
Filament winding	1.1
Resin transfer moulding	1.2
Hand lay-up	1.4
Hand-held spray application	2.2

(ii) Design Elastic Modulus of FRP (E_{fd})

Since the modulus of elasticity of FRP may change with time [53], it is necessary to apply a partial safety factor to this property too, in assessing the serviceability of structures strengthened with FRP. Recommended partial safety factors for modulus of elasticity (γ_{mE}) are given in Table 3.3 .

Table 3.3: Partial safety factor for modulus of elasticity at the ultimate limit state (γ_{mE}) [53]

Material	Factor of safety (γ_{mE})
Carbon FRP	1.1
Aramid FRP	1.1
Glass FRP	1.8

C. Properties of the Adhesive and Laminate Resin

It is important that the adhesive or laminating resin being used is compatible with the laminate or fiber. Ideally, to ensure compatibility, all the components of the system (including any priming or top coating materials) should be from a single supplier.

In general, the ultimate behavior of a strengthened section will be governed by the strength of the concrete and not by the strength of the adhesive, provided the following conditions are satisfied:

- All the materials used are in accordance with recognized standards.
- The material properties are checked on samples made on site.
- The in-service temperature does not differ significantly from that at which the test samples were made and cured.
- Detailed and proven method statements and specifications are used.
- Suitably qualified staff carries out the work.

- The structure is fail-safe, i.e. failure of the strengthening will not lead to failure of the structure.

If any of the above parameters are not satisfied, higher values of γ_{mA}, the partial safety factor for adhesive, will be required.

It should be noted that, cyclic strains applied to an adhesive during the curing period, for example due to traffic loading on a bridge under repair, may lead to a change in the properties of the adhesive. However, it has been suggested that these changes are likely to be small, perhaps a 10% reduction in the strength of the fully cured material. As a general recommendation, the sustained stress in the adhesive should be kept below 25% of the short-term strength, which equates to the recommended minimum material partial safety factor of 4.0 [53].

3.3 Design of the CFRP Strengthened Slab – Column Connection

The equations developed for the flexural design of FRP strengthened slabs are based on the rectangular parabolic stress-strain relationship for concrete and steel. Unlike steel reinforcement, all FRP reinforcement has a linear elastic response to failure, with no or very limited yielding. Woven fabrics have a degree of non- linearity, but this may be ignored for design purposes.

The flexural capacity will be determined using two methods, first the Elastic theory using the influence service line charts of (Reylonds) [41] and secondly the collapse load method using the yield line method.

The design for punching shear strengthened slab (the subject of this thesis) will be based on the papers:

i. Elstner and Hognestad (1995) [16].
ii. Harajli and Soudki (2003) [20].

3.3.1. Flexural Design of the CFRP Strengthened Slab –Column Connection

Adding FRP reinforcement to the tension side of the slab at the slab – column connection will increase the flexural capacity of the slab. In the following the flexural design for strengthened slab with singly reinforced section will be explain:

3.3.1.1 Bending Capacity of a Strengthened singly reinforced Section

The guidelines of the concrete society (UK) 2000 [53] to determine the moments of resistance Mr of the strengthened sections are based on the following assumptions:

- Sections that are plane before bending remain plane after bending.
- There is no slip between the FRP and the concrete.
- The stress-strain responses for concrete and steel reinforcement follow the Idealized curves presented in current codes and standards.
- FRP has a linear elastic response to failure.
- The tensile strength of the concrete is ignored.

There are three possible modes of failure depending on the ratio between the design ultimate moment due to applied loads, M, and the balanced moment of resistance of the section M_b; namely:

a. Balanced failure ($M = M_b$)

In a strengthened section occurs when the concrete and the FRP reach their ultimate design strains simultaneously; the depth of the neutral axis is x_b .

b. $M < M_b$

The FRP will reach its design tensile strain before the concrete crushes , However, failure normally occurs due to plate separation rather than plate rupture and the strain limits for debonding will frequently govern the design ,the depth of the neutral axis is less than the balance depth of the neutral x_b .

c. $M > M_b$

The concrete reach its design compressive strength before the FRP reaches its design tensile strain, the depth of the neutral axis is greater than x_b.

The detailed design for each of the above modes is carried out as follows:

a. Balanced Moment of Resistance

For a singly reinforced section with a layer of FRP bonded to the tension face, the strain and stress distribution will vary as shown in Figure (3.1) .The stress distributions shown are based on the BS 8110 models for concrete and steel. Taking moments about the bottom face, the moment of resistance for balanced failure, M_b

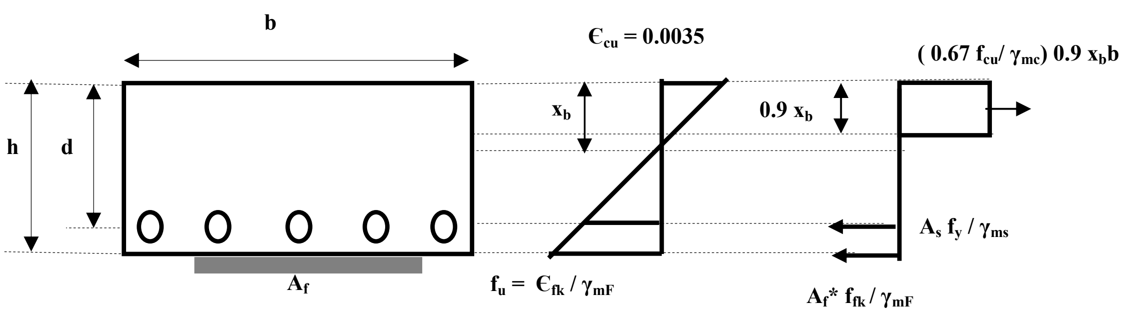


Figure (3.1): Single strengthened section stress and strain distribution for balance moment.

$$M_b = (0.67 f_{cu} / \gamma_{mc}) \, b \, 0.9 \, x_b \, [Z + (h - d)] - (f_y / \gamma_{ms}) \, A_s \, (h - d) \qquad (3.3)$$

$$x_b = h / (\epsilon_{fu} / \epsilon_{cu} + 1) \qquad (3.4)$$

$$Z = d - 0.45 \, x_b \qquad (3.5)$$

Where

M = design ultimate moment due to applied loads

M_b = balanced moment of resistance of the section taken about FRP

h = overall depth of beam, assuming thickness of FRP plate +adhesive « h .

ϵ_{fu} = design ultimate failure strain of FRP = $\epsilon_{fk} / \gamma_{mF}$

ϵ_{cu} = ultimate strain of concrete = 0.0035

f_{fk} = characteristic mechanical strength of FRP

Z = lever arm

x_b = depth of neutral axis for the balance failure

A_S = area of tension reinforcement

A_f = area of FRP

f_Y = ultimate stress of steel

f_{cu} = ultimate compressive strength of concrete

b = width of the section

d = effective depth

γ_{mc} , γ_{ms} , γ_{mF} = the partial safety factors for concrete , steel and FRP respectively.

b. M < M_b

The failure will be due to the FRP reaching its design tensile strain before the concrete crushes as shown in Figure (3.2). The design ultimate moment M is less than the balanced moment of resistance of the strengthened section M_b, the approximate area of FRP required, A_F, can be obtained by dividing the additional moment capacity required, M_{dd}, by the product of the steel lever arm, Z and the ultimate design stress in the FRP, f_{fd}, as follows:

$$M_{dd} = M - M_o \tag{3.6}$$

$$A_F = M_{dd} / f_{fd}\, Z \tag{3.7}$$

$$f_{fd} = \epsilon_{fu} * E_{fd} \tag{3.8}$$

$$E_{fd} = E_{fk} / \gamma_{mE} \tag{3.9}$$

$$\epsilon_{fu} = \epsilon_{fk} / \gamma_{mF} \tag{3.10}$$

Or the strain limit for the debonding, whichever is the lesser.

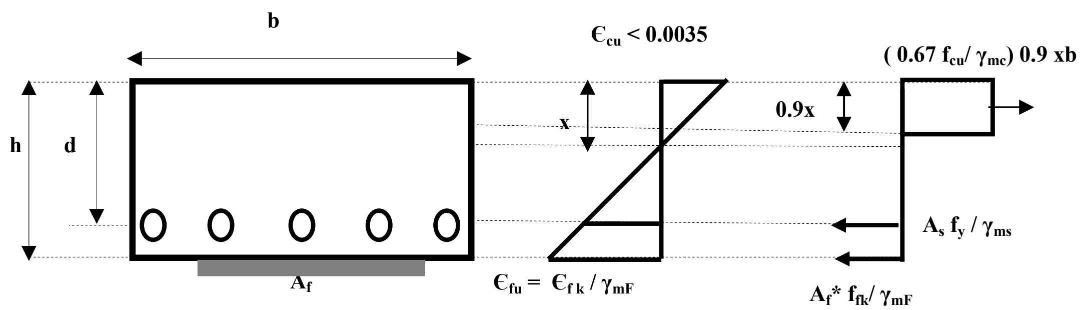


Figure (3.2): Single strengthened section stress and strain disruption for $M < M_b$

Taking moments about the bottom of the un-strengthened section

$$M_o = (0.67 f_{cu} / \gamma_{mc})\ b\ 0.9\ x\ [z + (h - d)] - (f_y / \gamma_{ms})\ A_s\ (h - d) \tag{3.11}$$

$$(0.67 f_{cu} / \gamma_{mc})\ b\ 0.9\ x = (f_y / \gamma_{ms})\ A_s \tag{3.12}$$

$$x = 1.66\ A_s f_y\ \gamma_{mc} / (f_{cu}\ \gamma_{ms}\ b) \tag{3.13}$$

Substituting for $\gamma_{mc} = 1.5$ and $\gamma_{ms} = 1.15$ gives:

$$x = 2.163\ f_y\ A_s / (f_{cu}\ b) \tag{3.14}$$

$$Z = d - 0.45\ x \tag{3.15}$$

Where

Mo = moment of resistance for un-strengthened section

x = depth of neutral axis

Note here x and Z are related for the un-strengthened section.

c. $M > M_b$

If the design moment exceeds the balanced moment of resistance of the strengthened beam, the failure will be due to the concrete crush as shown in figure (3.3). The design procedure is slightly more involved as both the tensile force in the FRP, F_r, and the tensile stress in the FRP, f_f, are unknown. The stress in the FRP will be less than the ultimate value and can be determined from the strain distribution in the member. Account should be taken of the initial strain in the concrete at the time of strengthening. The actual strain in the FRP is obtained by subtracting the initial strain, ϵ_{cit}, from the final strain in the concrete ϵ_{cft}, at level of the FRP based on a linear strain variation in the strengthened member.

The stress in the FRP is then calculated from:

$$f_f = E_{fd} * (\epsilon_{cft} - \epsilon_{cit}) \qquad (3.16)$$

$$E_{fd} = E_{fk} / \gamma_{mE} \qquad (3.17)$$

Taking moment about the bottom of the strengthened section

$$M = (0.67 f_{cu} / \gamma_{mc}) \, b \, 0.9 \, x \, [z + (h - d)] - (f_y / \gamma_{ms}) A_s (h - d) \qquad (3.18)$$

And substituting for x

$x = 2 * (d - z) / 0.9$ *and* $\gamma_{mc} = (3/2)$ *gives:*

$$M = (8/9)\, f_{cu}\, b\, (d\text{-}z)\, [z + (h - d)] - (fy / \gamma_{ms})\, A_s\, (h - d) \quad (3.19)$$

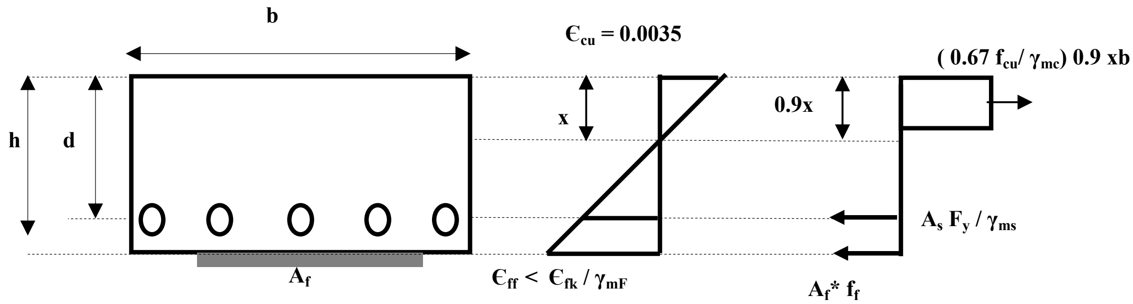


Figure (3.3): Single strengthened section stress and strain distribution for M > M_b

By substituting for the value of M due the applied load, z can be determined.

Taking moments about the compressive force in the concrete gives:

$$M = (f_y / \gamma_{ms})\, A_s\, z + F_r\, [z + (h - d)] \quad (3.20)$$

By substituting for the value of M due to the applied load and value of z from the equation (3.17) solve for F_r, therefore

$$A_f = F_r / f_f \quad (3.21)$$

The initial tensile strain in the concrete ϵ_{cit}, at the concrete / FRP interface can be calculated on the basis of an elastic analysis of the cracked section using the following

$$\epsilon_{cic} = M_S\, x_o / E_C\, I_{ce} \quad (3.22)$$

$$\epsilon_{cit} = \epsilon_{cic}\, (h - x_o) / x_o \quad (3.23)$$

$$f_f = E_{fd}\ (\epsilon_{cft} - \epsilon_{cit}) \tag{3.24}$$

Where

M_s = service moment based on the un-factored permanent loads acting on the un- strengthened member, Permanent loads include imposed load.

Where

ϵ_{ff} = strain in FRP

f_f = stress in FRP

Fr = tensile force in FRP

ϵ_{cic} = initial compressive strain in concrete.

ϵ_{cit} = initial tensile strain in concrete.

ϵ_{cft} = final tensile strain in the concrete

x_o = depth of neutral axis of un-strengthened transformed crack section.

E_c= modulus of elasticity of concrete.

I_{ce} = second moment of area of existing concrete equivalent transformed cracked section.

The equivalent transformed section may be obtained by assuming that the modular ratio of steel to concrete, $\alpha_s = E_S / E_C$ is 15.

3.3.1.2 Flexural Failure Load (F) in Slab–Column Connection:

The behavior of the slab–column connection under the shear force can be simulated by a two- way slab simply supported along four edges and acted upon by a concentrated “area load” (F) in the middle.

The most common methods of determining the bending moment (m) in such slabs under concentrically area load (F) which cause flexural failure are the “Elastic Method” and the “Yield Line Method”.

(1) Elastic Theory

The bending moment can be determined using the service influence line charts of [Reylonlds] [41], where the bending moment in a two way slab with a concentrically concentrated area load is given by

$$m = F\,(\alpha_{x4} + \upsilon\, \alpha_{y4}) \qquad (3.25)$$

Where

F = Total load on area to cause flexural failure.

m = bending moment per unit width -in units of the load (F) and the width

υ = poison ratio

α_{x4}, α_{y4} are factors from tables 54, 55 Reynolds [41]

(2) Yield Line Theory

For an isotropically reinforced square slab, simply supported along four edges and carrying a concentrated area load (F) at the middle; the expected yield –line pattern was shown in figure (3.4) in which x was an unknown dimension[16] .

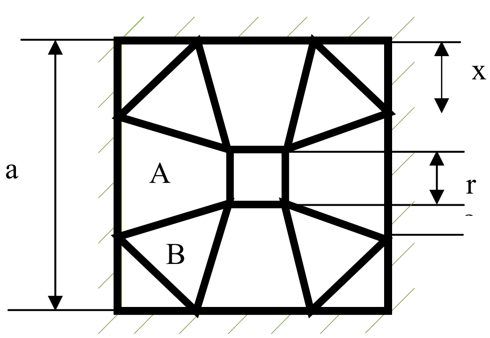


Figure (3.4) Expected yield line pattern for istropically simply supported slab with concentrated area load

Using the virtual work method

$$External\ work = F\ \delta \tag{3.26}$$

$$Total\ internal\ work = \sum (M\theta) \tag{3.27}$$

For element A :

$$Internal\ work\ W_A = m\ (a\text{-}2x)\ 2\ /\ (a\text{-}r) \tag{3.28}$$

For element B :

$$Internal\ work\ WB = m\ \surd\ 2\ x\ \surd 2\ /\ (a\text{-}r\text{-}x) \tag{3.29}$$

$$Total\ internal\ work = 4m\ [(a\text{-}2x)\ 2\ /\ (a\text{-}r) + \surd\ 2\ x\ \surd 2\ /\ (a\text{-}r\text{-}x)] \tag{3.30}$$

For unit external displacement and equating the internal and the external works

$$F = 4m\ [(a\text{-}2x)\ 2\ /\ (a\text{-}r) + \surd\ 2\ x\ \surd 2\ /\ (a\text{-}r\text{-}x)] \tag{3.31}$$

To obtain value of x giving minimum value of F, differentiating w.r.t.x:
$\partial F / \partial x = 0$

$$[-\ (4\ /\ (a\text{-}r)) + (2\ (a\text{-}r\text{-}x) + 2\ x)\ /\ (a\text{-}r\text{-}x)^{2}] = 0$$

This is solved for x as follows:

$$4\ /\ (a\text{-}r) = 2\ (a\text{-}r)\ /\ (a\text{-}r\text{-}x)^{2}$$

$(a-r-x)^{2} - (a-r)^{2} / 2 = 0$

$x^2 - 2\,(a-r)\,x + (a-r)^{2}/2 \;\; = 0$

$x = (1- \sqrt{2} / 2)\,(a - r)$

Substituting for the value of x in equation (3.22) gives:

$$F = 4m\,[(a-2(1- \sqrt{2} / 2)\,(a - r))\,2 / (a-r) + \sqrt{2}\,(1- \sqrt{2} / 2)\,(a - r)\sqrt{2} / (a-r-(1- \sqrt{2} / 2)\,(a - r))]$$

$$= 8m\,[a / (a-r) - (2- \sqrt{2}) + (1+ \sqrt{2} / 2)\,(a - r) /((a-r- a + r)+\sqrt{2} / 2)\,(a - r))]$$

$$= 8m\,[a / (a-r) - (2- \sqrt{2}) + (1- \sqrt{2} / 2) / \sqrt{2} / 2]$$

$$F = 8m\,(1/ (1- r/a) - 3 + 2\sqrt{2}) \qquad (3.32)$$

Where

F= the concentrated area load to cause flexural failure.

M = Total internal moment along yield line.

r = the width of the column.

a = the width of the slab.

3.4 Punching Shear Strengthening Of the Slab – Column Connection:

The punching shear capacity equation suggested by the BS 8110 is:

$$V_c = [(0.79)\,(100\,As / (b_v\,d)\,)^{(1/3)}\,(400 /d\,)^{(1/4)}\,(f_{cu} /25\,)^{(1/3)}/1.25]*u*d \qquad (3.33)$$

Where

V_c = punching shear capacity of the slab

$A_s / (b_v\, d)$ = the reinforcement ratio.

u = perimeter at 1.5 d.

As = tensile steel reinforcement.

f_{cu} = ultimate compressive strength of concrete

b_v =width of the section of maximum shear force.

d =effective depth.

This equation cannot be used to predict the punching shear capacity of the strengthened section, because it does not include the additional FRP reinforcement.

Unfortunately, up to now, there is no expression in the available guidelines for the punching shear capacity of slabs strengthened with FRP.

Several researchers [14, 35, 46, 49] concluded that the enhancement of the flexural capacity of the slab will lead to an enhancement in the punching shear capacity of the slab. To find expression for punching shear capacity for the strengthened slab, a relation between the flexural capacity and the punching capacity of the slab must be established.

Several experimentally and analytically based expressions have been proposed to evaluate the punching shear capacity of slabs and footing [54] . Most of these expressions recognized the influence of flexural strength on the punching shear capacity of the slab.

One of the available design expressions in which the flexural strength capacity is considered a parameter in calculating the punching shear strength of the slab is the equation proposed by **Mowrer and Vanderbilt [35]**, in which the punching shear capacity P_u is given as

$$P_u = \frac{0.8\,(1 + d/r)\, b_o\, d\, f_c^{\wedge 0.5}}{(1 + 0.433\, b_o\, d\, f_c^{\wedge 0.5} / P_{flex})} \tag{3.34}$$

Where

P_u = punching shear resistance

b_o = perimeter of column

f_c = concrete cylinder compressive strength = $0.8f_{cu}$

P_{flex} = load applied to cause flexural yield

r = width of the column

This equation shows that the increase in the flexural capacity will lead to an increase in the punching shear capacity. So for the flexural strengthened slab there will be an enhancement in the punching shear capacity.

3.5 FRP Separation Failure

Members strengthened externally with FRP can fail prematurely as a result of local FRP separation. This can be caused by two different mechanisms namely: peeling and debonding.

Peeling failure often occurs at the ends of the FRP plate where there is a discontinuity as a result of the abrupt termination of the plate. It is normally associated with concentrated shear and normal stresses in the adhesive layer due to the FRP deformation that takes place under load. The magnitude of these stresses is influenced by various factors including the dimensions of the FRP plate, the mismatch in the modulus of elasticity of the FRP and the adhesive, and the shape of the bending moment diagram. Peeling failure usually results in ripping off the concrete cover along the level of the internal steel reinforcement, towards the centre of the member.

Unlike peeling, debonding normally occurs away from the plate end, debonding is generally associated with the formation of wide flexural and shear cracks that occur as a result of the yielding of the embedded steel bars. The wide cracks generate high stresses in the FRP across the crack, which can only dissipate by debonding. This debonding can then propagate towards the plate end, leading to FRP separation failure. Also use of weak adhesive, improper application and inadequate preparation of the concrete substrate can lead to premature debonding.

Plate separation is a controversial topic and the preceding descriptions are intended to provide a brief overview of some of the factors involved [5]. Early research on FRP separation failure suggested a number of possible approaches for combating this problem, including:

- The use of plate end anchorage devices.
- The use of flexible adhesives.
- Imposing limits on the plate aspect ratio (i.e. breadth/thickness ratio).

Bolted systems, bonded angle sections and composite straps bonded across the soffit of the plates are examples of plate end anchorage devices that have been proposed as possible methods of preventing FRP separation failure. However, none of these have proved to be entirely satisfactory and they are generally regarded as ineffective or impractical for normal use, particularly for slabs [53].

Although laboratory tests have demonstrated that flexible adhesives reduce the risk of premature plate separation, producing adhesive formulations that are flexible and yet have sufficient strength and stiffness to transfer shear force between the plate and concrete has proven to be difficult.

Work on bonded steel plates showed that there is a significant relationship between plate aspect ratio and risk of peeling failure. It was further revealed that with aspect ratios greater than 50, premature failures can be avoided. This finding was subsequently incorporated in the U.K. Highways Agency Advice Note on steel plate bonding, BA 30 [6]. Unfortunately, there is presently insufficient test data on bonded FRP plates to establish an appropriate aspect ratio for these materials. All that can be said at this stage is that, since FRP plates are normally applied in very thin sheets, FRP plates with aspect ratios far greater than 50 should be acceptable.

As stated above, plate separation is a controversial topic; much research is being directed towards the determination of the precise mechanisms involved. However, it must not be forgotten that the present rules are semi- empirical and their extrapolation to cases different from those upon which they are based must be undertaken with caution.

Nevertheless, the result of further work suggests that if the simplified adequate procedure for analysis and design is adopted, a conservative solution to the problem of FRP separation will result. It is hoped that as more data becomes available, some relaxation of the design limits assumed in the procedures will be possible.

Laboratory tests on plated beams have shown that the incidence of peeling failure will reduce if the increase in the tensile force in the FRP plate is gradual. Since the FRP plate will usually be terminated in the tension zone, the shear stress at the FRP/concrete interface should be kept to a minimum. For simply supported members, this condition will be achieved by stopping the FRP as close to the support as possible.

Accordingly end plate separation failure can be avoided by addressing two criteria:

1. Limiting the longitudinal shear stress between the FRP and the substrate.
2. Anchoring the FRP by extending it beyond the point at which it is theoretically no longer required.

3.6 Serviceability

Loads should not adversely affect the appearance or efficiency of strengthened structures. Generally, for the FRP strengthened structures, the cracks width should not exceed the limits recommended in BS 8110 and the steel reinforcement should not yield under the service load; otherwise permanent deformations in the structure will result. Fatigue and stress rupture can be controlled by using lower design stresses.

3.6.1 Crack Widths

In normal cases, crack widths will not be excessive providing the FRP strengthening system has been properly installed. Where uncertainty exists, however, it should be verified that crack widths at service loads do not exceed the limits recommended in BS 8110. Guidance on calculating crack widths in reinforced concrete structures is given in Section Three of BS 8110: Part 2. This method can be adapted for FRP strengthened structures by taking into account the transformed area of the FRP laminate in calculating the stress in the tension steel. The second moment of area of the section should be determined assuming that the long-term modular ratios of steel to concrete, α_s, and FRP to concrete, α_f, are given by

$$\alpha_s = Es / (E_c / 2) \tag{3.35}$$

$$\alpha_f = E_{fd} / (E_c / 2) \tag{3.36}$$

It is worth noting that, in reality, calculating crack widths is not as straightforward as suggested here. This is because, as the FRP strengthening system is placed on the surface, the spacing of the cracks and hence the crack width is significantly reduced for the same strain. However, no detailed work defaming the extent of this reduction in crack width has been carried out. In the interim, therefore, the procedure outlined above, though was conservative, is recommended.

3.6.2 Deflections and Material Stresses

Deflections can be controlled according to BS 8110 Part 1: clause 5.4.6.1.1 by limiting the span / depth ratio.

Long and short -term deflections can be calculated as per BS 8110 part 2 provisions.

The assumptions used in the analysis are as follows:

- Strains are calculated on the basis that plane section remains plane.
- The reinforcement is elastic with a modulus of elasticity of 200 kN/mm^2
- The concrete in compression is elastic.
- The modulus of elasticity of the concrete to be used is the mean value given in BS 8110 : part 2, table 7.2

For the singly reinforced partially cracked section shown in figure 3.5:

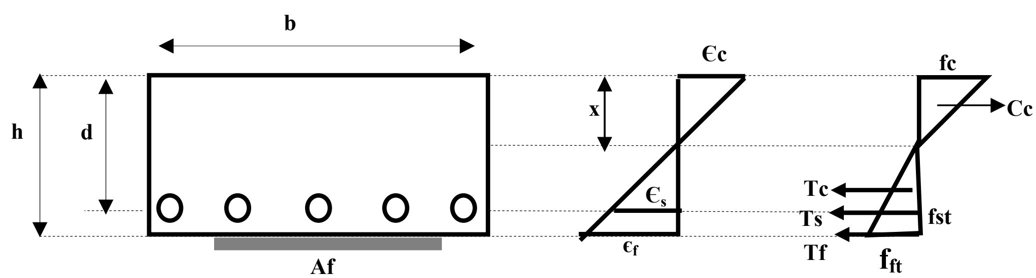


Figure (3.5): Partially cracked section stress and strain distribution

To find the depth of neutral axis x, take moments about the neutral axis of the given section and transform the area of the steel and CFRP laminate using the given values of α_s and α_f respectively to obtain:

$$bx^2/2 = \alpha_s As\ (d-x) + \alpha_f A_f (h-x) \qquad (3.37)$$

$$b\, x^2/2 + (\alpha_s As + \alpha_f A_f)\, x - (\alpha_s As\, d + \alpha_f A_f h) = 0 \qquad (3.39)$$

$$x = \frac{-(\alpha_s As + \alpha_f A_f) + \sqrt{(\alpha_s As + \alpha_f Af)^2 + 2b(\alpha_s As\, d + \alpha_f A_f h)}}{b} \quad (3.40)$$

The tensile force in the concrete T_c is given by:

$$Tc = 0.5 f_{ct} (h- x)^2 / (d-x) \quad (3.41)$$

Where

f_{ct} is the tensile stress in concrete at the level of the tension reinforcement.

The tensile moment of resistance of the concrete is

$$M_{ten} = 2\, T_c\, (h - x) / 3 \quad (3.42)$$

Thus the net moment M_{net} will be

$$M_{net} = M - M_{tent}\ , \quad (3.43)$$

Where

M is the applied moment.

The moment of inertia of the transformed section about the N.A axis I_{eq} is obtained as:

$$I_{eq} = 1/3\ bx^3 + \alpha_s As\, (d-x)^2 + \alpha_f A_f\, (h - x)^2 \quad (3.44)$$

Hence, the curvature is given by

$$1/r = M_{net} / Ec\, I_{eq} \quad (3.45)$$

The deflection (a) is calculated from

$a = \; k\, l^2\, (1/r)$ (3.46)

In which k is a constant that depends on the shape of bending moment diagram, given in Table 3.1 BS 8110 : part 2 For the concentrated load at mid span k = 1/ 12

The existing stress

$б_c = M_{apply} / I_{eq}\, x \leq \; б_c$ *allowable* (3.47)

$б_s = \alpha_s\, M_{appy} \;/ I_{\,eq}\; (\, d\text{-}x) \leq б_s$ *allowable* (3.48)

$б_f = \alpha_f\, M_{apply} / {}_{I\,eq}\; (h\text{-}\; x) \leq б_f$ *allowable* (3.49)

Where

T_c , T_S ,T_f = tensile force in the concrete ,steel and FRP respectively.

f_c = stress of concrete in compression.

f_{st} = stress of steel in tension .

f_{ct} = stress of concrete in tension

A_s = tensile steel area

x = depth of the N.A

h = depth of the beam

d = effective depth

b = width of the section

l = span of the beam

r = radius of curvature

a = deflection

C_c = force in the concrete in compression

E_S = modulus of elasticity of steel

E_c = modulus of elasticity of concrete

α_s , α_f = modular ratio for steel and CFRP respectively

3.6.3 Stress Rupture

Rupture of the FRP may occur at service loads due to the sustained stresses that exist in the material. Therefore it is recommended that the maximum stress in the FRP at service loads, as a proportion of the design strength, should not exceed the values given in Table 3.4 [53] .

Table 3.4: Maximum stress under service loads to avoid stress rupture as proportion of design strength (%) [53]

Material	Maximum stress (%)
Carbon FRP	65
Aramid FRP	40
Glass FRP	55

3.7 Summary

Design of the FRP strengthened slab – column connection bases on the limit state principles.

For the ultimate limit state the slab- column connection must be check for

- Bending:

The load which causes flexural failure was determined using two methods:

i. Elastic method.

ii. Yield line theory.

- shear:

The punching shear capacity will be determine using two formulas:

i. Shear capacity formula suggested by BS8110 to verify for its accuracy.

ii. Mowrer and Vanderbilt [35] , in which the flexural capacity of the connection was considered as one of the parameters .

- FRP separation failure

To prevent plate separation failure there were suggested number of approaches such like; use plate end anchorage devices, or extend the FRP beyond the point at which is required, use flexible adhesive, and imposing limit for plate aspect ratio.

For serviceability limit state the strengthened slab –column connection, it was recommended to:

- Control the crack with as recommended by BS 8110.
- Control the deflection as recommended byBS8110 by limiting the span/ depth ratio, the long term deflection can be determine as per BS8110, but the formulas should be adjusted that the FRP into taken into consideration .
- Reinforcement should not yield under service load to prevent permanent deformation.
- Fatigue and stress rupture can be controlled by using lower design stresses.

CHAPTER 4

Experimental Specimens and the Test Procedures

4.1 Description of the Test Specimens:

Two types of flat slabs specimens with column stub at the middle to represent the interior slab – column connection were used in this experimental investigation. Both were basically thin square plates of the same dimensions and both are orthogonally reinforced. One was reinforced with 9 T10 and the other by 9T12 each way.

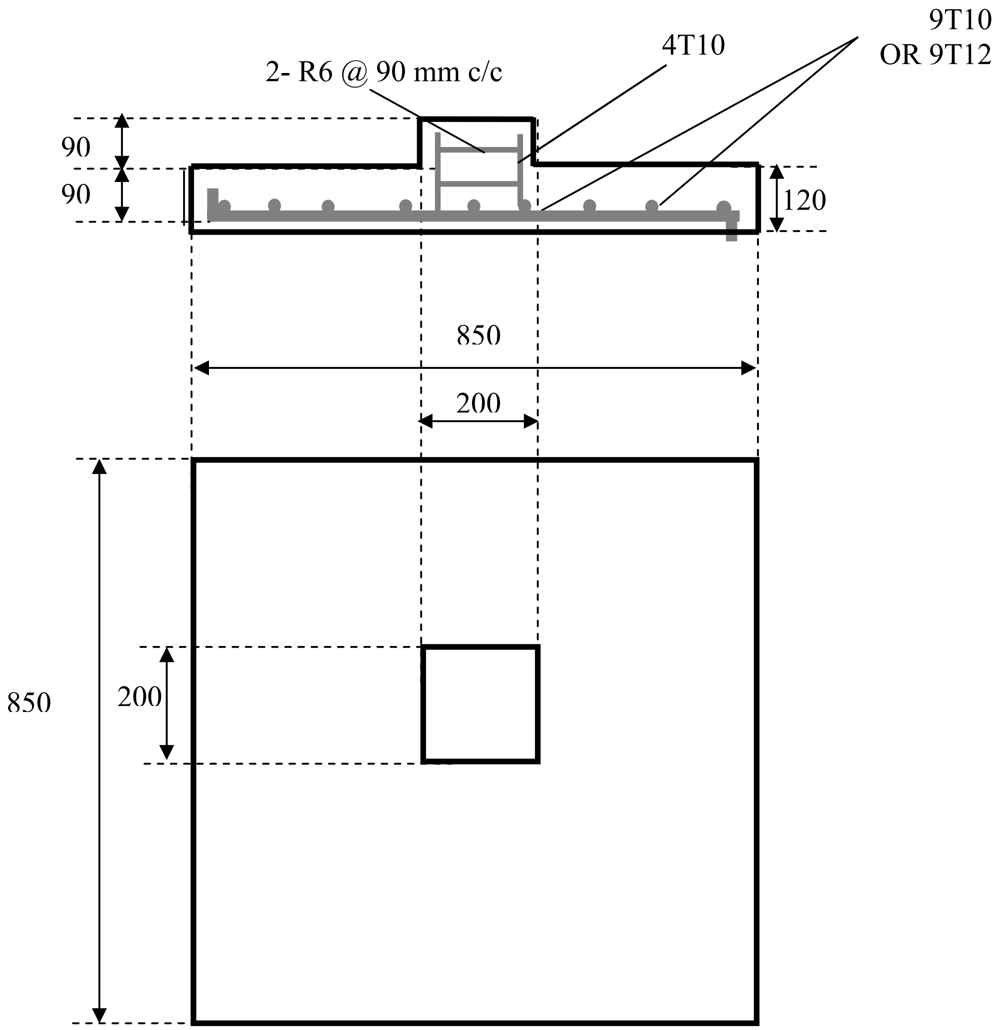
4.2 Characteristics of the Specimens:

4.2.1 Specimens Dimensions:

The test specimens consisted of six square slabs of 850 X 850 X120 mm with a central square column stub 200 X 200 mm of 90 mm height to represent the interior slab–column connection. The column stub was cast monolithic with the slab as presented in figure (4.1).

4.2.2 Reinforcement detail:

The specimens are divided into two series (A & B) according to the reinforcement details. Series A is reinforced with 9 T 10@100 mm c/c both ways and series B with 9 T12@100 mm c/c both ways as per figure (4.1), casting and curing is explain in Figure(4.2).



all dimensions in mm

a. Typical dimensions of the slab

b. Reinforcement cage

c. Reinforcement cage in formwork

Figure (4.1) Structural details of test specimen

Figure (4.2): Casting and curing process

4.2.3 Materials Used:

The aggregates used were natural sand and crushed stone as fine and coarse aggregate respectively.

The sand was locally available, free from impurities and silts (< 4%), and complying with the limits of the BS 882:1992. Appendix A-1 shows the grading of sand.

The Coarse aggregate is crushed stone with maximum size of 10 mm and crushing value of 19%. Appendix A -2 shows the grading of coarse aggregate.

The tension reinforcements were high tensile deformed bars of 10 mm diameter for series A, and 12 mm diameter for series B. The stub was reinforced with four vertical plain bars of diameter 8 mm diameter, and two links of 6 mm diameter. All the reinforcements were tested according to the BS 4449 and the results are given in Appendix A-4.

The cement used in concrete mix was ordinary cement manufactured by Gena factory for series A, and by Atbra factory for series B. The test results for both types of cements according to BS 12:1996 are presented in Appendix A-3.

All of the slabs have the same concrete mix with a target compressive strength 30 Mpa, the concrete mix design was presented in Appendix A-7.

Each series of the slabs together with 9 standard 100*100*100 cubes were casted using one concrete batch .The results of compressive strength of the cubes were given in Appendix A-8.

The strengthening material for the all specimens was the carbon fiber reinforced polymer, which was Sika Carbodur Pultruded Carbon Fiber Plate; the specifications of which are given in Appendix A – 5. The plates were bonded to the tension face of the slab in two directions parallel to the directions of steel reinforcement.

The Adhesive used to bond the CFRP plates to the concrete surface was the epoxy sikadur -30 given in Appendix A-6 .

4.2.4 Design of the Control Specimens:

The control specimens were designed to fail in shear, the flexural capacity was determined by Elastic theory and yield line theory, while the punching capacity was determined by the BS8110-1987equation's. The equations suggested by Mowrer and Vanderbilt [35] were also used to calculate punching shear capacity because many researchers verified that the BS equation's are conservative.

Thus the design of the specimens was carried out as follows:

1. Specimen A:

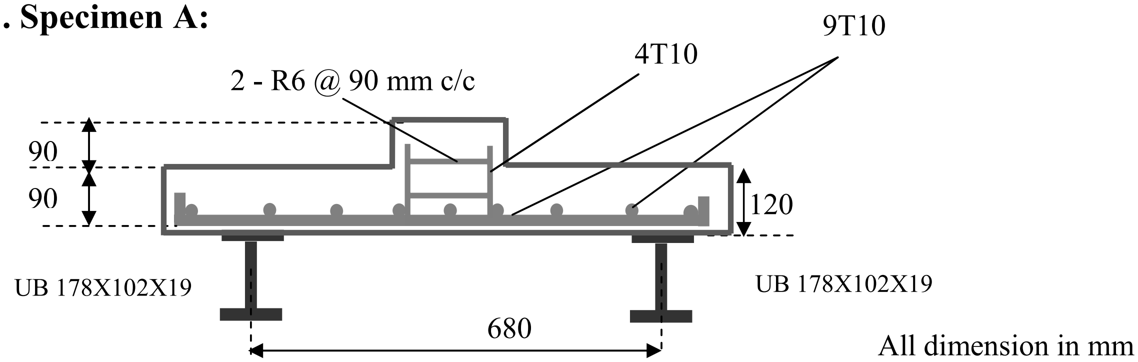


Figure (4.3): Structural details for specimen A

$L_{c/c}$ = 680 mm (lx, ly) , h = 120 mm , A_{st} = 7 Φ10 (549.8 mm^2) ,

f_{CU} = 50.7 Mpa (Appendix A-8), f_y = 450.6 Mpa (Appendix A – 3) , r = 200 mm

Flexural Capacity Of specimen A:

The Collapse load based on the flexural capacity of the slab can be estimated by either of the two following methods:

i. Elastic Analysis

Using Equation (3.25) of Chapter Three the total load to cause flexural failure is:

$F = m / (\alpha_{x4} + \upsilon\, \alpha_{y4})$

$d = 120 - 10 - 20 \qquad = 90$ mm

$\rho_s = 549.8 / (90 * 680) \qquad = 0.009$

$m = \rho_s f_y d^2 \{1 - 0.45 (2.164 \rho_s f_y / f_{cu})\}$

$= 0.009 * 450.6 * 90 ^\wedge 2 * \{ 1 - 0.45 * 2.164 * 0.009 * 450.7 / 50.7$

$= 30.287$ kN.m/m

To find α_{x4} ,α_{y4} from table 55 Reynolds

$K = l_x / l_y = 680 / 680 \qquad = 1$

$a_x = a_y = r + 2d$

$= 200 + 2*90 \qquad = 380$ mm

$a_x/l_x = 380 / 680 \qquad = 0.56$

$a_y / l_y = 380 / 680 \qquad = 0.56$

Therefore from figure for k =1

$a_x / l_x = 0.56$,

$a_y / l_y = 0.56 \quad \alpha_{x4} = \alpha_{y4} \qquad = 0.083$

$F = m / (\alpha_{x4} + \nu\, \alpha_{y4})$

$= 30287 / (0.083 + 0.2 * 0.083) \qquad = 304.09$ k N

ii. Yield Line Theory

Using equation (3.32) of Chapter Three

$F = 8\, m\, (1 / (1 - r/a) - 3 + 2\sqrt{2})$

$F = 8 * 30.287 * (1 / (1-200/680) - 3 + 2\sqrt{2}) \qquad = 301.678$ kN

<u>Punching Shear Capacity of specimen A:</u>

The Collapse load based on the punching shear capacity of the slab can be estimated by either of the two following methods:

i. Empirical Equation BS 8110 1985

Equation (3.33) of Chapter Three is used to calculate the punching shear is:

$V_c = 0.79\, (100\, As / b_v\, d)^{(1/3)}\, (400 / d)^{(1/4)}\, (f_{cu} / 25)^{(1/3)} * u * d$

$= 0.79\, (0.009 * 100)^{(1/3)}\, (400 / 90)^{(1/4)}\, (50.7 / 25)^{(1/3)} * (12*90+4*200)*90$

$= 237.2$ kN

ii . Mowrer and Vanderbilt Equation

Equation (3.34) 0f Chapter Three, the punching shear capacity is:

$$P_u = \frac{0.8\,(1 + d / r)\; b_o\, d\, f_c^{\wedge 0.5}}{(1 + 0.433\, b_o\, d\, f_c^{\wedge 0.5} / P_{flex})}$$

$$= \frac{0.8(1+ 90 / 200) * 800 * 90 * (0.8*50.7/)^{\wedge 0.5}}{(1+ 0.433 * 800 *90 *(0.8*50.7/)^{\wedge 0.5} / 301678)} \qquad = 320.789 \text{ kN}$$

2. Specimen B:

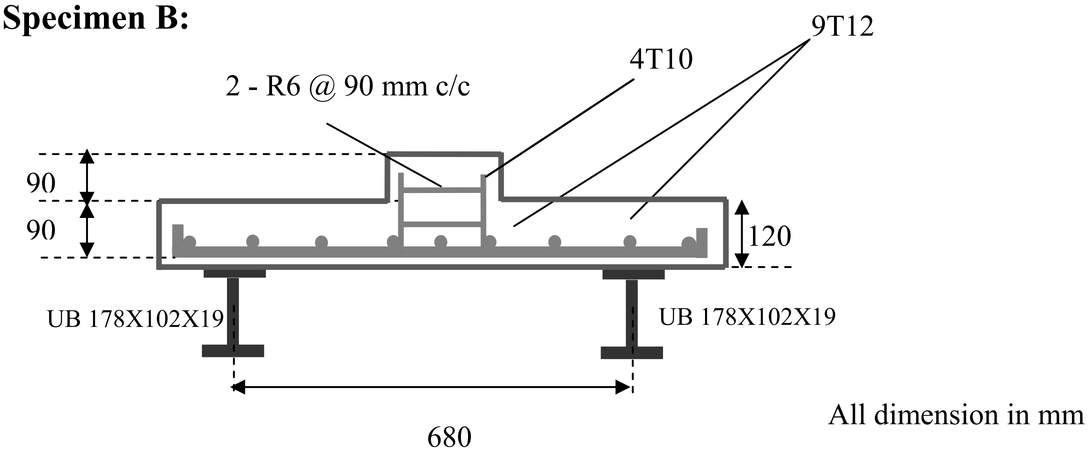


Figure (4.4) Structural details for specimen B1

$L_{c/c}$ = 680 mm (lx, ly) , h = 120 mm, A_{St} = 7 Φ12 (791.6 mm^2) ,
f_{CU} = 44 Mpa (Appendix A-8), f_y = 473 Mpa **(**Appendix A– 3) , r = 200 mm

Flexural Capacity of specimen B

i. Elastic Theory

Using Equation (3.25) of Chapter Three, the total load to cause flexural failure is:

$$F = m / (\alpha_{x4} + \upsilon\, \alpha_{y4})$$

$d = 120 - 12 - 20 \qquad = 88$ mm

$\rho_s \quad = 791.6 / (88 * 680) \qquad = 0.013$

$m \ = \ \rho_s f_y d^2 \{1 - 0.45 (2.164 \rho_s f_y / f_{cu} \ )\}$

$= 0.013 * 473 * 88 \hat{} 2 * \{1 - 0.45 * 2.164 * 0.013 * 473 / 44\}$

$= 41.138$KN.m/

To find α_{x4} , α_{y4} from table 55 Reylond

$K = l_x / l_y = 680 / 680 = 1$

$a_x = a_y = r + 2d$

$= 200 + 2*88 \qquad = 376$ mm

$a_x / l_x = 376 / 680 \qquad = 0.57$

$a_y / l_y = 376 / 680 \qquad = 0.57$

Therefore from the figure for k =1

For

$a_x / l_x = 0.55$,

$a_y / l_y = 0.55 \quad \alpha_{x4} = \alpha_{y4} = 0.085$

$F \quad = m / (\alpha_{x4} + v\ \alpha_{y4})$

$= 41138 / (0.085 + 0.2 * 0.085) \qquad = 403.31$ k N

ii. Yield Line Theory

Using equation (3.32) of Chapter Three, the load to cause flexural failure is :

$F = 8\ m\ (1 / (1 - r / a) - 3 \ + 2 \sqrt{2})$

$F = 8*41138 *(1 / (1-200/680) - 3 + 2 \sqrt{2}) \qquad = 409.77$ kN

Punching shear capacity of specimen B

i. Empirical Equation BS 8110- 1985

Equation (3.33) of Chapter Three is used to calculate the punching shear is:

$$V_C = 0.79\ (100\ As\ /\ b_v\ d)^{(1/3)}\ (400\ /d)^{(1/4)}\ (f_{cu}\ /25)^{(1/3)}\ *u*d$$

$$= 0.79\ (0.013*100)^{(1/3)}\ (400\ /88)^{(1/4)}\ (44\ /25)^{(1/3)}\ *(12*88+4*200)*88$$

$$= 248.3\ kN$$

ii. Mowrer and Vanderbilt Equation

Equation (3.34) of Chapter Three the punching shear capacity is:

$$P_u = \frac{0.8\ (1 + d\ /\ r)\ b_o\ d\ f_c^{\wedge 0.5}}{(\ \ 0.433\ b_o\ d\ f_c^{\wedge 0.5}\ /\ P_{flex})}$$

$$= \frac{0.8\ \ (1+ 88\ /\ 200)\ *800\ *88*(0.8*44)\ \hat{}0.5}{(1 + 0.433\ *800*88\ *(0.8*44)\ \hat{}0.5\ /\ 409770)} = 336.2\ kN$$

4.2.5 Strengthening schemes

BS 8110 states that the shear reinforcement shall be between 0.5d and 1.5d from the column face, accordingly the positions of CFRP plates were proposed to be located within the same range for the shear reinforcement.

Specimens A-2&B-2 were strengthened with 4 CFRP plates of 50 mm width symmetrically placed, in the four directions, at distance 65 mm from the face of column – thus the CFRP plates were put in the middle of the distance recommended by the BS 8110 for the shear reinforcement. These two slabs constitute the first strengthening scheme shown in Figure (4.4).

Specimens A-3&B-3 were strengthened with 8 CFRP plates, each two plates placed adjacent to each other to give a width of 100 mm, at the face of column symmetrically placed in the four directions. These two slabs constitute the second strengthening scheme shown in Figure (4.5).

Specimens A-1& B-1 were left without strengthening as control specimens.

.

(a) First strengthening scheme

(b) Second strengthening scheme

Figure (4.5): Strengthening schemes

Test parameters, specimen designations, and relevant information pertaining to each specimen are provided in Table 4.1.

Table 4.1: Test Parameter

series	Specimen designation	Slab depth (mm)	Steel reinforcement	CFRP plate width (mm)	Distance of the edge of plate from the face of column (mm)	Concrete strength f_{cu} (Mpa)	Reinforcement Strength (Mpa)
A	A1	120	9T 10	_	_	50.7	450.6
	A2	120	9 T 10	50	65	50.7	450.6
	A3	120	9 T 10	100	0	50.7	450.6
B	B1	120	9 T 12	_	_	44	473
	B2	120	9 T 12	50	65	44	473
	B3	120	9 T 12	100	0	44	473

4.3 Strengthening Procedure:

4.3.1 Surface Preparation

All the proposed positions were grinded using grinding hammer to remove the mortar layer and expose the coarse aggregates to achieve good bonding between the adhesive and the concrete. Then the surface was cleaned with wire brush and hover to remove all small particles as shown in Figure (4.6).

a. Grinding and grinding tools.

b. Surface cleaning with the brush and hover

Figure (4.6): Concrete surface preparation

4.3.2 CFRP Plate Surface Preparation

The CFRP plates were cut to the required length 850 mm using mechanical cutter, and the surface on each sides of the plate was cleaned with sika colma cleaner and white cloth to remove contaminates and carbon dust . The process, presented in Figure (4.7), was repeated until there is no change in the colour of the white cloth.

(a) Cutting of the CFRP plates

(b) Surface cleaning of the CFRP plates

Figure (4.7): CFRP Surface preparation

4.3.3 Adhesive Mixing

The epoxy adhesive consists of two components, A with white colour, and B with black colour, to be mixed in the ratio of A:B= 3:1 by weight. First each material was stirred in its original container and then the recommended amount of component B was added to that of A. The two components were mixed together using a low speed electric mixer (480 rpm) for about three minutes to reduce the entrained air. Figure (4.8) illustrates this process.

a. Mixing component A

b. Mixing component B.

d. Weighting component B

e. weighting component A

f. Mixing the two components

Figure (4.8): Adhesive mixing

4.3.4 Application of strengthening

The well-mixed sikadur adhesive is first applied to the cleaned substrate in a convex shape with plastering techniques to give a thickness of 1-2 mm, and then applied to the cleaned carbodur laminate. The coated laminate is then placed onto the coated concrete surface and pressed with a rubber roller until the adhesive is forced out on both sides of the plate. The excess adhesive is to be removed with a scraper.

To insure good bonding at the intersection of the plates, the surface of the first plate is cleaned with the colma cleaner before applying the adhesive.

Figure (4.9) illustrate this process.

a. Application of epoxy to substrate

b. Application of epoxy to CFRP plate

c. Pressing of CFRP laminates to slab

d. Strengthening scheme 1

e. Strengthening scheme 2

Figure (4.9): Installation process

4.4 Test Set Up

The test set up and the experimental procedures are the same for all specimens. Each specimen was mounted onto a square frame 770 X770 mm made of UB 178X102X19, thus the specimen extended 40 mm beyond the centre lines of the UBs. This represented a simply supported condition with the corners free to uplift. Pieces of 6 mm plates were welded to the top of the UB with gaps in positions of the CFRP plates to prevent pressing them by the slab during the loading process.

The square frame was placed diagonally on two opposite stiff I beams; thus each member of the UB frame members was supported at its middle as shown in figure (4.10)

The load was applied by hydraulic jack concentric with the column stub. A dial gauge to measure the deflection at the middle was put under the slab at the middle. The load was applied at 3 tons increments until failure.

At the end of each test, the distance at which the shear cracks appeared away from the column face was measured and the crack pattern for each specimen was carefully examined.

Loads and deflections readings are presented in Appendix A.9

a. Square frame on the stiff beams

b. Test set up

c. the hydraulic machine reading

d. Dial gauge

Figure (4.10) Test set up and readings instruments

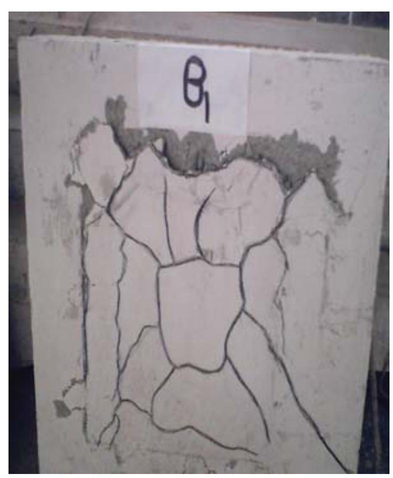
CHAPTER (5)
ANALYSIS OF RESULTS AND CONCLUSION

5.1 Failure Modes

5.1.1 Control Slabs:

Each slab was loaded until failure. The cracks started with flexural cracks along the perimeter of the column, extending from the corners of the column in a diagonal direction and then forked out as they approached the corners of the slab. The cracks extended excessively until punching shear cracks appeared around the stub column at distance of about 2d from the face of column and finally the slab failed in a clear punching shear mode with the column stub penetrating the slab from the tension side as shown in photo of Figure (5.1). The failure load for both specimens $A_1$and B_1 was 340 kN; which is much higher than the predictions of BS8110 equation.

a. Crack pattern at failure for slab A1



b. Crack pattern at failure for slab B1

Figure (5.1): Typical crack pattern for control specimens

The deflections values in slab B_1 was less than in slab A_1, which show clearly that the increase in the reinforcement ratio affects the flexural capacity more than the punching shear force, see figure (5.2) .

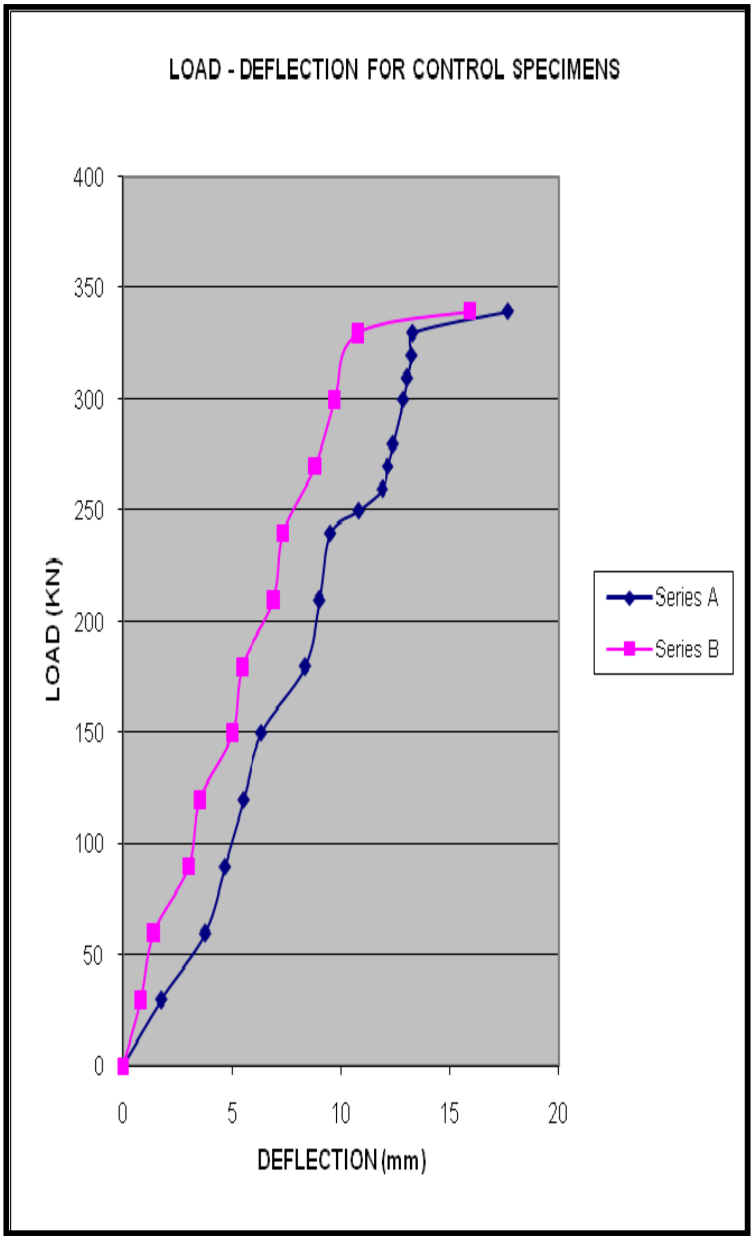


Figure (5.2): Load versus deflection for the control specimens

5.1.2 Strengthened slabs:

Series A:

Unfortunately Slab A-2 and A-3 showed premature failure due to the debonding of the CFRP plates at the ends of the slab with the same punching shear failure load of 340 kN of the control specimens.

Cracking in specimen A_2 started with flexural cracks around the column but the CFRP plates prevented them from progressing to the corners of the specimen. Punching shear cracks appeared at distance of about 2d from the face of column and as the load reached 340 kN, debonding at the ends of the plates occurred and the specimen suddenly failed by punching shear.

In slab A-3 no flexural cracks were observed until the punching cracks appeared at the same distance of 2d from the face of column, followed by failure in an identical manner to A-2.

The load- deflection curve for series A shows that the use of CFRP plates reduces the deflections value significantly for specimen A2, and A3.

Figure (5.3) shows the failure modes while Figure (5.4) shows the load-deflection curves for the specimens of series A.

Figure (5.3): Crack pattern at failure for series A

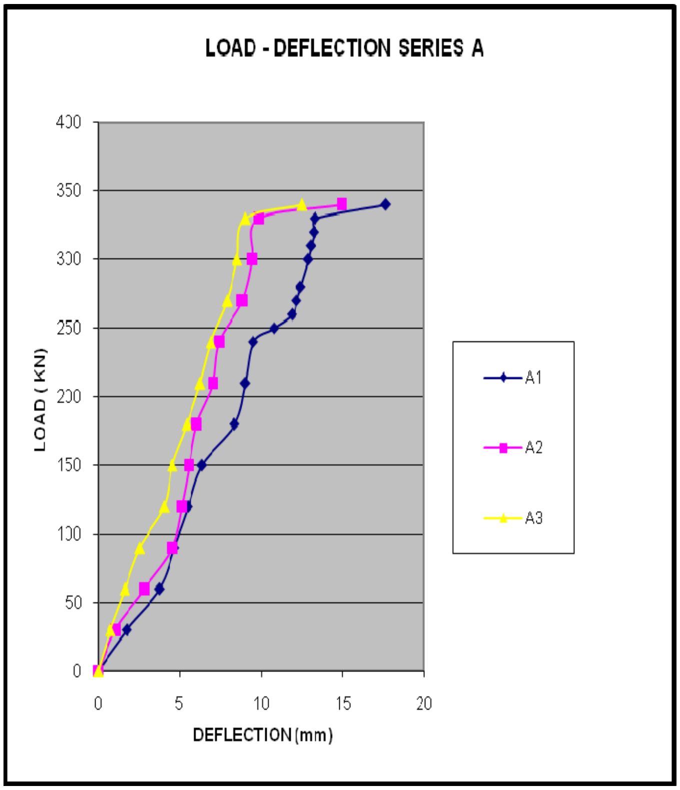


Figure (5.4): Load versus deflection for series A

Series B:

Slabs B-2 and B-3 also showed premature failure due to the debonding of the CFRP plates at their ends.

Slab B-2 behaved in the same manner as slab A-2 but with higher loads for the first flexural and punching shear cracks.

For Slab B-3 no cracks were observed until the debonding of plates took place, followed by punching shear failure.

Also here the load- deflection curve for series B shows that the use of CFRP plates reduces the deflections value significantly for specimen B2, and B3. Both slabs failed at the same load of 340 kN.

Figure (5.5) shows the failure while Figure (5.6) shows the load-deflection cures for the specimens of series B.

Figure (5.5): Cracks pattern at failure for series B

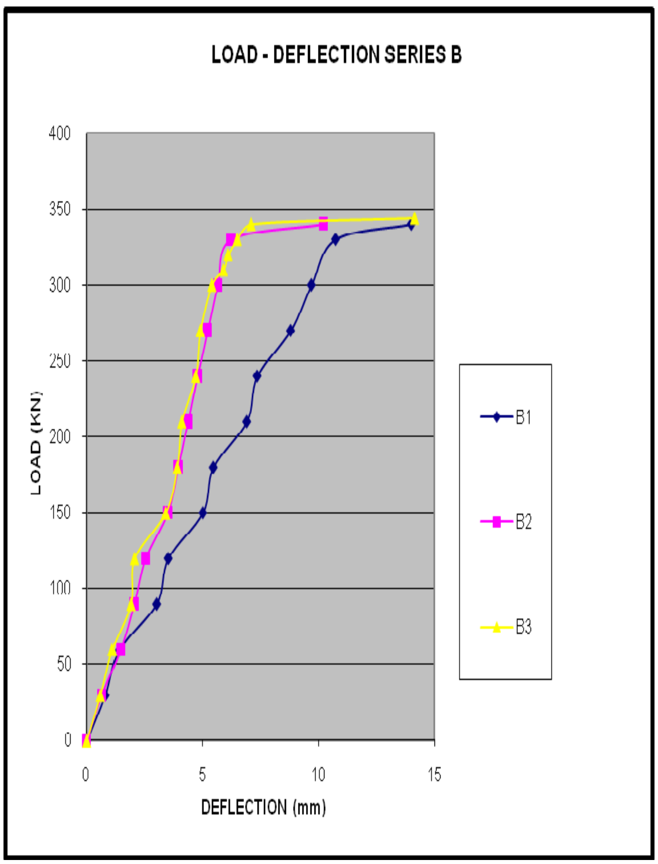


Figure (5.6): Load versus deflection for series B

5.2 Summary and Conclusion:

Six two–way square slabs with a 90 mm high column stub at the centre were tested under static concentrated load through the column stub to induce punching shear failure The dimensions of the slabs were 850x850x120 mm and those of the column stub were 200x200mm.

Four of the slabs were strengthened with CFRP laminates in two different schemes:

In the first scheme single strips were bonded to the tension face of the slab at a distance of d/2 from the column faces in the four directions.

In the second scheme two strips-placed side by side- were bonded to the tension face of the slab at column faces in the four directions.

The remaining two specimens – the control specimens –were left un strengthened.

The comparison between theoretical & experimental failures is presented in Table (5.1) below.

Table (5.1) Theoretical& experimental failure load

Specimen	Theoretical Flexural capacity		Theoretical Punching shear capacity		Experimental failure load (KN)
	Elastic theory (KN)	Yield line theory (KN)	equation by BS8110 (KN)	Mowrer and Vanderbilt (KN)	
A	304.09	301.6	237.2	320.79	340
B	403.31	409.77	248.3	336.2	340

The following conclusions are made

- The equation of punching shear capacity recommended by the BS 8110, 1985 for unstrengthened slab [clause 3.5.3] is conservative.

- The proposed equation by Mowrer and Vanderbilt to determine the punching shear capacity for unstrengthed slabs shows good agreement with the experimental values for the control specimens.

- Although the use of CFRP plates to strengthen slabs for punching increases the stiffness and delays the formation of cracks, it promotes the brittle mode of failure.

- For the range covered by this study the debonding failure is the governing Mode of failure for the strengthened slabs.

- First strengthen scheme (single strip bonded to the tension slab face at distance d/2) is more suitable for punching shear enhancement than the second one (two strips-placed side by side bonded to the tension slab face at column faces in the four directions) because the second scheme increases the brittle mode of failure.

- The use of CFRP laminates to strengthen slabs for punching shear is hindered by the debonding problem.

5.3 **Recommendations for future studies:**

- The ease of application of CFRP laminates to strengthen slabs for punching shear justifies further research on the debonding problem.

- Further suggestions for suitable devices to prevent debonging failure.

- Further investigation for the ductility behavior for the strengthened systems to prevent brittle failure mode.

- Further seeking for other formulas for punching shear capacity for slabs.

- Extension of the proposed slab – column connection experimental model to simulate the realistic flat slab system.

- Use of first proposed strengthened scheme was more suitable than the second one, as the second strengthening scheme enhancement the brittle mode of failure.

- On basis of the experimental results of the tested specimens the debonding failure can be expected to happen at load more than 80% of the total load capacity.

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A.1 Sand Tests

Sieve Analysis

Table A.1 Sand sieve analysis Results

BSS	R	%R	%P
10	4	1	99
5	10	2.4	97.6
2.36	25	6	94.0
1.18	71	16.9	83.1
0.6	164	39.0	61.0
0.3	272	64.8	35.2
0.15	390	92.9	7.1
TOTAL (PAN)	420	100	0.0

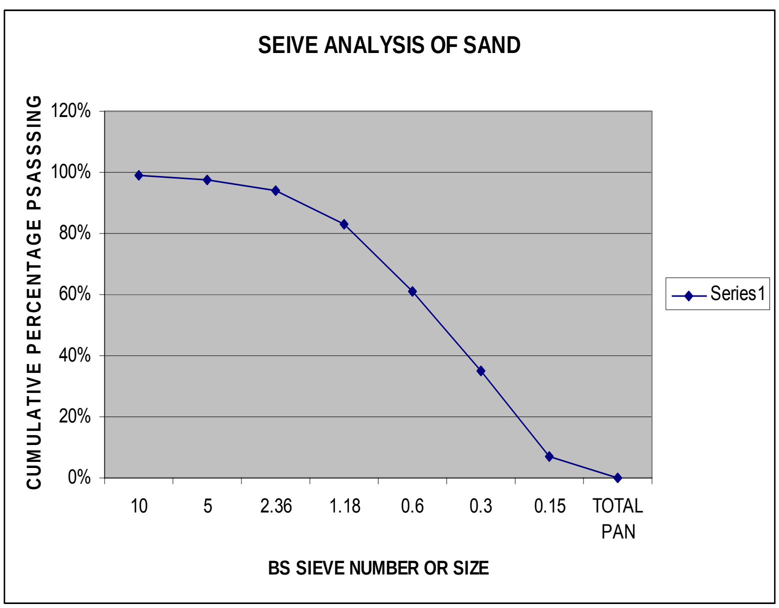

Figure (A.1) Sand sieve analysis curve

Sand type: medium fine (see BS 882: 1992, table 4)

Where
R: cumulative retained weight per grams
%R: percentage of retained weight
% p: percentage of passing weight
Total: total weight of sample per grams
BSS; British standard sieves
Dust, silt and clay content: < 4% ok

A.2 Aggregate tests:

Table A.2 Course Sieves analysis

BSS	R		%P
50.0	0	0	100
37.5	0	0	100
20.0	0	0	100
14.0	0	0	100
10.0	183	10.5	89.9
5.0	1564	85.9	14.7
total	1820	100	0

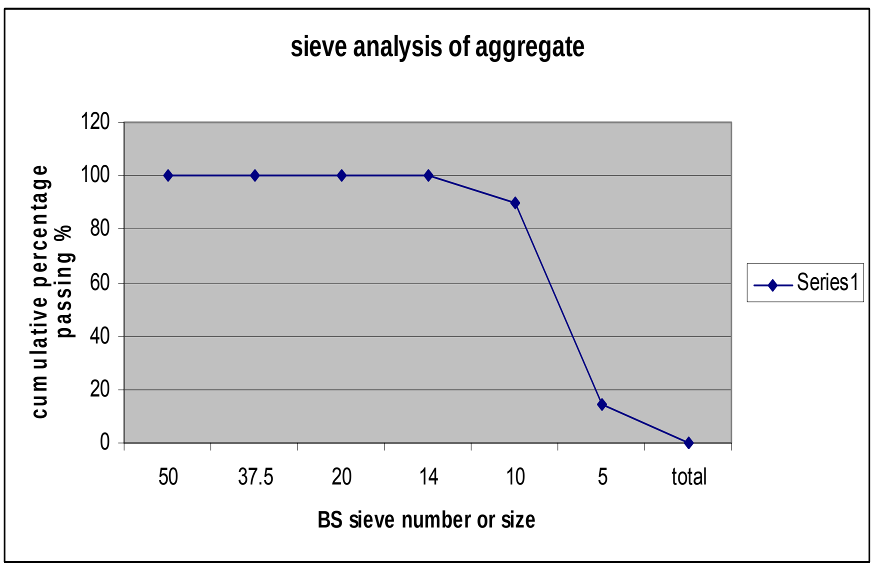

Figure (A.2) Course sieve analysis curve

The tested sample complied to 10 mm single size aggregate
*Crushing value = 19 %
*Absorption = 1.6 %

A.3 Cement

a. Atbara cement

Test (NO)	Test	Result	Requirement of BS12:1996
1	Consistency	28 %	-
2	Setting time a. Initial b.Final	 2:15 2:55	 Not less than 60 min Not more than 10 hrs
3	Compressive strength 2 days 1 2 3 28 days 1 2 3	 17.3 18.6 18.6 32.2 30.5 34.1	Equal to or greater than 10 N/mm^2

b. Gena cement

Test (NO)	Test	Result	Requirement of BS12:1996
1	Consistency	27 %	-
2	Setting time a. Initial b. Final	 2:00 2:50	 Not less than 60 min Not more than 10 hrs
3	Compressive strength 2 days 1 2 3 28 days 1 2 3	 20.1 20.8 21.7 47.3 47.6 47.9	Equal to or greater than 10 N/mm^2

A.4 .1 Reinforcing steel Φ 10 (deformed)

Specimen Designation		Diameter (mm)		Effective Cross Section Area (mm2)	Weight Kg / ml	Strength N/ mm^2		Fu/Fy	Elongation %
		Nominal	effective			yield	ultimate		
1	1	10	10.18	81.5	0.640	434	691	1.59	20
	2		10.02	78.8	0.616	456	708	1.54	22
	3		10.01	78.7	0.618	462	703	1.52	19

A.4.2 Reinforcing steel Φ 12 (deformed)

Specimen Designation		Diameter (mm)		Effective Cross Section Area (mm2)	Weight Kg / ml	Strength N/ mm^2		Fu/Fy	Elongation %
		Nominal	effective			yield	ultimate		
1	1	12	11.94	112.1	0.880	470	631	1.34	27
	2		11.94	112.1	0.88	480	640	1.33	27
	3		11.97	112.5	0.883	496	642	1.29	23

A.5 Sika carbodur plates product sheet

Product Data Sheet
Edition 11, 2006
Version no. 04.06

Construction

Sika Carbodur® Plates

Pultruded Carbon Fiber Plates for Structural Strengthening

Product Description	Sika Carbodur® Plates are pultruded carbon fiber reinforced polymer (CFRP) laminates designed for strengthening concrete, timber and masonry structures. Sika Carbodur® Plates are bonded onto the structure as external reinforcement using Sikadur® -30 for normal - or Sikadur® -30LP epoxy resin for elevated application temperatures (for details on the adhesive see the relevant Product Data Sheet).
Uses	To strengthen structures for: <u>Load increase</u> ■ Increasing the capacity of floor slabs and beams ■ Increasing the capacity of bridges to accommodate increase axle loads ■ Installation of heavier machinery ■ Stabilising vibrating structures ■ Changes of building use <u>Damage to structural elements</u> ■ Deterioration of original construction materials ■ Steel reinforcement corrosion ■ Vehicle impact ■ Fire ■ Earthquakes <u>Service improvements</u> ■ Reduced deflection ■ Stress reduction in steel reinforcement ■ Crack width reduction ■ Reduced fatigue <u>Change in structural system</u> ■ Removal of walls or columns ■ Removal of slab sections for openings <u>Change of specification</u> ■ Earthquakes ■ Changed design philosophy <u>Design or construction defects</u> ■ Insufficient / inadequate reinforcement ■ Insufficient / inadequate structural depth.
Advantages	■ Non corrosive ■ Very high strength ■ Excellent durability ■ Lightweight ■ Unlimited lengths, no joints required ■ Low overall thickness, can be coated ■ Easy transportation (rolls) ■ Simple plate intersections or crossings ■ Very easy to install, especially overhead ■ Outstanding fatigue resistance

- Minimal preparation of plate
- Combinations of high strength and modulus of elasticity available
- High alkali resistance
- Clean edges without exposed fibers thanks to the pultrusion process
- Approvals from many countries worldwide

Approval / Standards

Deutsches Institut für Bautechnik Z-36.12-29, 2002: General Construction Authorisation for Sika Carbodur®.

SOCOTEC Rapport No. HX0823, 2000: Rapport d'enquete technique / cahier des charges - Sika Carbodur® / Sika Wrap® (French).

NBI Teknisk Godkjenning, NBI Technical Approval, No. 2178, 2001, (Norwegian).

ZAG, Technical Approval No. S418/99-620-2, za uporabo nacina ojacitev armirano betonskih in prednapetih elementov konstrukcij z dolepljenjem lamel iz karbonskih vlaken "Sika Carbodur®" v Republiki Sloneniji (Slovenian).

TSUS, Building Testing and research institutes,
Technical approval No. 5502A/02/0633/0/004, 2003: Systém dodatocného zosilnovania zelezobetonovych a drevenych konstrukcil Sika Carbodur® (Slovak).

Instytut badawczy drog i mostow, technical approval No. AT/2003-04-0336, System materialow Sika Carbodur® do wzmacniania konstrukcji obiektow mostowych (Polish).

Fib, Technical Report, bulletin 14: Externally bonded FRP reinforcement for RC structures, July 2001 (International).

ACI 440.2R-02, Guide for the Design and construction of Externally Bonded FRP Systems for strengthening concrete structures, October 2002, (USA).

Concrete Society Technical Report No. 55, Design guidance for strengthening concrete structures using fiber composite material, 2000 (UK).

SIA 166, Klebebewehrungen, 2003 /2004 (CH).

Product Data

Sika Carbodur® CFRP Plates

Appearance / Colour

Carbon fiber reinforced polymer with an epoxy matrix, black.

Packaging

Cut to size according parts list in packaging.
Supplied in rolls of 250 m in packing boxes.

Types

Sika Carbodur® S — Tensile E-Modulus 165'000 N/mm²

Type	Width	Thickness	Cross Sectional Area
Sika Carbodur® S512	50 mm	1.2 mm	60 mm²
Sika Carbodur® S612	60 mm	1.2 mm	72 mm²
Sika Carbodur® S812	80 mm	1.2 mm	96 mm²
Sika Carbodur® S1012	100 mm	1.2 mm	120 mm²
Sika Carbodur® S1512	150 mm	1.2 mm	180 mm²
Sika Carbodur® S914	90 mm	1.4 mm	126 mm²
Sika Carbodur® S1014	100 mm	1.4 mm	140 mm²
Sika Carbodur® S1214	120 mm	1.4 mm	168 mm²

Sika Carbodur® M (Steel Equivalent) — Tensile E-Module > 210'000 N/mm²

Type	Width	Thickness	Cross Sectional Area
Sika Carbodur® M614	60 mm	1.4 mm	84 mm²
Sika Carbodur® M914	90 mm	1.4 mm	126 mm²
Sika Carbodur® M1214	120 mm	1.4 mm	168 mm²

Sika Carbodur® H — Tensile E-Module > 300'000 N/mm²

Type	Width	Thickness	Cross Sectional Area
Sika Carbodur® H514	50 mm	1.4	70 mm²

Storage Conditions / Shelf Life

Unlimited (no exposure to direct sunlight, dry).

Technical Data

Density	1.60 g/cm3
Temperature Resistance	> 150°C
Fiber Volume Content	> 68% (type S)

Mechanical / Physical Properties

		Sika Carbodur S	Sika Carbodur M	Sika Carbodur H
E-Modulus*	Mean Value	165'000 N/mm²	210'000 N/mm²	300'000 N/mm²
	Min. Value	> 160'000 N/mm²	> 200'000 N/mm²	> 290'000 N/mm²
	5% Fractile-Value	162'000 N/mm²	210'000 N/mm²	-
	95% Fractile-Value	180'000 N/mm²	230'000 N/mm²	-
Tensile Strength*	Mean Value	3'100 N/mm²	3'200 N/mm²	1'500 N/mm²
	Min. Value	> 2'800 N/mm²	> 2'900 N/mm²	> 1'350 N/mm²
	5% Fractile-Value	3'000 N/mm²	3'000 N/mm²	-
	95% Fractile-Value	3'600 N/mm²	3'900 N/mm²	-
Strain at break* (min value)		> 1.70%	> 1.35%	> 0.45%
Design strain**		0.85%	0.65%	0.25%

* Mechanical values obtained from longitudinal direction of fibers.
** These values should be used for design as the maximum strains in the CFRP-plates must be adapted to local design regulations as necessary. Dependent upon the structure and the load situation, they may also have to be decreased by the responsible Engineer according to requirements and standards.

System Information

Sika Carbodur® + Sikadur® -30 or Sikadur® -30LP

Application Details
Consumption

Width of Plate	Sikadur® -30
50 mm	0.35 kg/m'
60 mm	0.40 kg/m'
80 mm	0.55 kg/m'
90 mm	0.70 kg/m'
100 mm	0.80 kg/m'
120 mm	1.00 kg/m'
150 mm	1.20 kg/m'

Depending on the surface plane, profile and roughness of the substrate as well as any plate crossings and loss or wastage, the actual consumption of adhesive may be higher.

Substrate Quality

Evenness / plane or level: (according to FIB14)
The surface to be strengthened must be levelled, with variations and formwork marks not greater than 0.5 mm. Plane and level of the substrate to be checked with a metal batten. Tolerance for 2 m length max. 10 mm and for 0.3 m length 4 mm. These tolerances shall be adapted to local guidelines if there are any. They might be more restrictive.

Substrate strength (concrete, masonry, natural stone) must be verified in all cases): Mean adhesive tensile strength of the prepared concrete substrate should be 2.0 N/mm², min. 1.5 N/mm². If these values can not be reached, then see the Sika Wrap® Fabric Product Data Sheets for alternative Sika® solutions.
Concrete must be older than 28 days (dependent on environment and strengths).

Substrate Preparation	Concrete and Masonry: Substrates must be sound, dry, clean and free from laitance, ice, standing water, grease, oils, old surface treatments or coatings and all loosely adhering particles. Concrete must be cleaned and prepared to achieve a laitance and contaminant free, open textured surface. Repairs and levelling must be undertaken with structural repair materials such as Sikadur® -41 repair mortar or Sikadur® -30 adhesive, filled max. 1 : 1 by weight with Sikadur® -501 quartz sand. If levelling has been conducted more than 2 days before applying the plates, the levelled surface has to be grind again to ensure a proper bond between Sikadur® -41 and Sikadur® -30 (see the relevant Product Data Sheets). Timber surfaces: Must be prepared by planing, grinding or sanding. Dust must be removed by vacuum. Steel surfaces: Must be prepared by blastcleaning to Sa 2.5 free from grease, oil, rust and any other contaminants which could reduce or prevent adhesion Use primer (see table). Be careful to avoid water condensation (dew point). Priming can be done with Icosit® -277 or with Sikagard® -63N as temporary corrosion protection; or Icosit® -EG1 as permanent corrosion protection.

	+10°C	+20°C	+30°C
1) Maximum waiting time between - Blastcleaning of steel and - Primer / or Sikadur® -30 (application without priming possible, if no corrosion protection is needed)	48 hours	48 hours	48 hours
2) Minimum waiting time between - Primer and - Sikadur® -30 application (without additional preparation of the Primer)	48 hours	24 hours	12 hours
3) Maximum waiting time between - Primer and - Sikadur® -30 application (without additional preparation of the Primer)	7 days	3 days	36 hours
4) Waiting time between - Primer and - Sikadur® -30 application (with additional preparation of the Primer)*	> 7 days	> 3 days	> 36 hours

* If additional preparation of the primer is necessary (4), it shall be done at the earliest, the day before application. After preparation of the Primer, the surface has to be cleaned/vacuumed free from dust.

Plate preparation:
Immediately prior to the application of Sikadur® -30, solvent wipe the bonding surface with Sika® Colma Cleaner to remove contaminants. Wait until the surface is dry before applying the adhesive.

Application Conditions

Substrate Temperature	See the Product Data Sheets of Sikadur® -30 and Sikadur® -30LP.
Ambient Temperature	See the Product Data Sheets of Sikadur® -30 and Sikadur® -30LP.
Substrate Humidity	See the Product Data Sheets of Sikadur® -30 and Sikadur® -30LP.
Dew Point	See the Product Data Sheets of Sikadur® -30 and Sikadur® -30LP.

Application Instructions

Mixing	See the Product Data Sheets of Sikadur® -30 and Sikadur® -30LP.
Mixing Time	See the Product Data Sheets of Sikadur® -30 and Sikadur® -30LP.

Application Method	Place the Sika Carbodur® plate on a table and clean the unlabelled side with Colma Cleaner using a white rag. Apply the well-mixed Sikadur® -30 adhesive with a special "dome" shaped spatula onto the cleaned Carbodur® laminate. Apply the Sikadur® -30 adhesive carefully to the properly cleaned and prepared substrate, with a spatula to form a thin layer. Within the open time of the adhesive, place the Sikadur® -30 coated Sika Carbodur® plate onto the Sikadur® coated concrete surface. Using a Sika® rubber roller, press the plate into the adhesive until the material is forced out on both sides of the laminate. Remove surplus adhesive. Intersections / multiple layers: Where there are to be plate intersections or crossovers, the first Sika CarboDur® plate should be degreased with Sika® Colma Cleaner before overlaying with adhesive and then the second plate applied. If more than one plate is to be bonded together, they all have to be cleaned on both sides with Sika® Colma Cleaner - Use Sikadur® -330 or Sikadur® -30 adhesive in these instances (for details see the Product Data Sheets of Sikadur® -330 and Sikadur® -30). Quality assurance: Samples must be made up on site for quality control of curing rate and strength. Average standard values after curing 7 days at +23°C are: ■ Compressive strength > 75 N/mm² ■ Flexural tensile strength > 35 N/mm² These values can differ by up to 20% dependent on the circumstances. The following are the most important factors which can have a negative influence on the mechanical properties: ■ Air entrapment in the sample (from mixing or filling into the mould!) ■ Curing temperature / time ■ Contamination of the adhesive! Therefore care should be taken to avoid these situations. Application Tools: Sika® Colma Cleaner: For cleaning of Sika Carbodur® plate before bonding, cleaning of application tools. In 1 and 5 kg pails, 20 kg mini drum and 160 kg drum. Sika Carbodur® Rubber Roller: For pressing the Sika Carbodur® plate onto the surface. Sales unit 1 pce. Sika® Mixing Spindle: For minimizing air entrapment. Sales unit 1 pce.
Cleaning of Tools	Clean all tools and application equipment with Sika® Colma Cleaner immediately after use. Cured material can only be mechanically removed.
Pot Life	See the Product Data Sheets of Sikadur® -30 and Sikadur® -30LP.
Notes on Application / Limitations	A suitably qualified Engineer must be responsible for the design of the strengthening works. **This application is structural and great care must be taken in selecting suitably experienced and trained specialist labourers.** Only apply plates within the open time of Sikadur®-30. Site quality control should be supported/monitored by an independent testing authority. Care must be taken when cutting plates. Use suitable protective clothing, gloves, eye protection and respirator. The Sika Carbodur® system must be protected from permanent exposure to direct sunlight. Maximum permissible service temperature is approx. +50°C Note: When using the Sika CarboHeater® together with Sikadur® -30LP this can be increased to max. +80°C (see the Sika CarboHeater® Product Data Sheet). The instructions in the Technical Data Sheet must be followed when applying Sikadur® -30 adhesive.

Fire Protection	**If required Sika Carbodur® plates may be protected with fire resistant material. When the Sikadur® -30 has cured, test for voids by tapping the surface of the plate with metallic object or impuls-thermography.** Coating: The exposed plate-surface can be painted with a coating material such as Sikagard® -550W Elastic or Sikagard® -ElastoColor W.
Notes	All technical data stated in this Product Data Sheet are based on laboratory tests. Actual measured data may vary due to circumstances beyond our control.

Safety Instructions

Ecology	Small quantities of cured material may be burned in a municipal incinerator by agreement with authorities. Uncured components may not be discharged into drains, waterways or the ground.
Transport	Comp. (A): Non-hazardous. Comp. (B): 8/65 c), free quantity: 500 kg.
Safety precautions	Apply barrier cream to hands and unprotected skin before starting work. Wear protective clothing (gloves, safety glasses). In contact with eyes or mucous membranes rinse immediately with clean warm water and seek medical attention without delay.
Toxicity	Comp. (A): Class 4, under the relevant Swiss health and Safety codes. Observe warning on packing. Comp. (B): Non-Toxic.

Legal notes

Construction

Sika Egypt for Construction Chemicals
El Abour City
1st industrial zone (A)
Section # 10 Block 13035,
Egypt

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ISO 9002 ISO 14001

A.6 sikaduar -30 product sheet

Product Data Sheet
Edition 11, 2006
Version no. 01.03

Construction

Sikadur® -30

Adhesive for Bonding Reinforcements

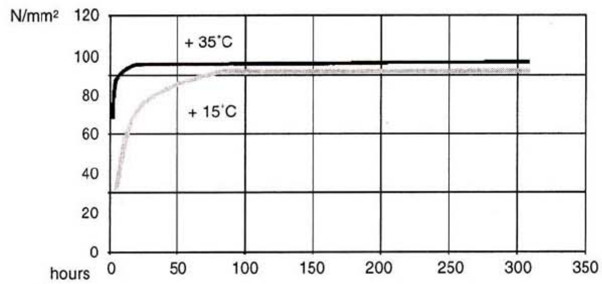
Product Description	Solvent-free, thixotropic, epoxy-based two-component adhesive mortar
Uses	As an adhesive for bonding reinforcements, adhesive mortar and plastic filler to: ■ Concrete, stone. ■ Steel. ■ Epoxy. For structural bonding of: ■ Sika® CFRP laminates to concrete. ■ Sika® CFRP laminates to timber. ■ Steel plates to concrete. ■ Concrete elements. ■ Bridge segments. ■ Kerbstones to concrete, etc. For bonding of: ■ Starter bars. ■ Wall anchors. ■ Fixings, etc. For vertical and overhead filling of: ■ Holes. ■ Dimensional inaccuracies etc.
Advantages	■ Long pot life. ■ Long open time. ■ Can be applied to slightly damp concrete surfaces. ■ Non-sag in vertical and overhead applications. ■ Good temperature resistance ■ Sets without residual tack, even in high atmospheric humidity. ■ Solvent free. ■ Rapid curing, even at low temperatures. ■ High temperature resistance. ■ High creep resistance under permanent load. ■ High mechanical strength. ■ High abrasion and shock resistance. ■ Shrinkage-free curing. ■ Easy to mix and apply. ■ Components come in different colours therefore homogeneity of mix is easy to check.
Product Data	
Colour	Comp. (A): white. Comp. (B): black. Comp. (A + B) mixed: light grey.
Consistency	Comp. (A + B) mixed: creamy paste
Packaging	5 kg units (A+B) pre-measured Large-size packing on request.
Storage	Store at temperatures between + 5°C and + 25°C.
Shelf Life	12 months from date of production if stored in original unopened packing.

Technical Data

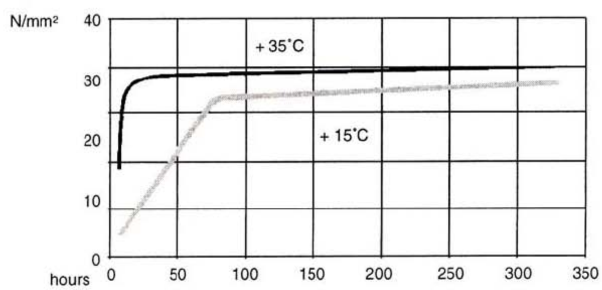
Density	1.77 kg/l.
Thixotropy (to Sika®)*	20 mm film thickness (at + 35°C).
Sag Flow (to F.I.P.)*	3 - 5 mm.
Squeezability at 15 kg (to F.I.P.)*	4 '000 mm^2 (at +15°C).
Shrinkage (to F.I.P.)*	0.04%.
Brittle Temperature (to F.I.P.)*	62 °C.
Modulus Of Elasticity (static)	12'800 N/mm^2.
Coefficient Of Thermal Expansion (as Sika®)	9×10^{-5} per °C (- 10°C to + 40°C).
Tensile Bending Strength (to F.I.P.)*	Concrete failure (4 N/mm^2).
Shear Strength (to F.I.P.)*	Concrete failure (15 N/mm^2).
Note	The figures given may vary according to the mixing intensity and the amount of air introduced.

*Fédération Internationale de la Précontrainte.

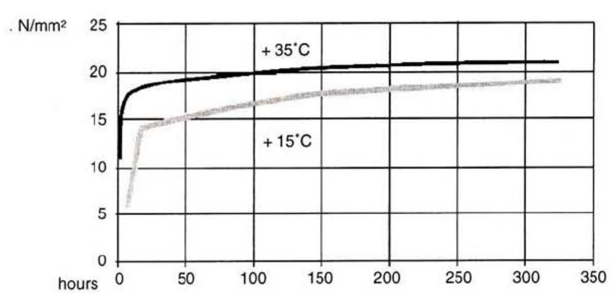
Compressive Strength



Tensile Strength



Shear Strength



Application

Mix Ratio	Comp. (A+B) =3:1 by weight and volume.
Substrate	Concrete. stone: ■ Clean, free from oil and grease, dry, no loose particles or laitance. ■ Concrete age, depending on climate, 3 to 6 weeks. ■ Preparation: Sandblast, high-pressure water jet. ■ If applied on damp concrete, work Sikadur® -30 well into the surface. ■ Max. substrate moisture:10%. ■ Min. adhesive strength of concrete substrate: 1.5 N/mm^2. ■ If the concrete surface has large uneven sections or holes after preparation, these must first be filled with Sikadur® -41. Timber: ■ Clean, free from oil and grease. ■ Sandblast or grind well. Steel: ■ Free from grease and oil, free from rust, scale and rolling "skin". ■ Preparation: Sandblast SA 2.5. ■ Beware of condensation (dew point). ■ If the cleaned steel is not bonded immediately, the surface must be given one coat of Sikagard® -62 to protect it from further corrosion. Consumption: approx. 0.3 kg/m^2. Epoxy: ■ Free from oil and grease. ■ Grind well using coarse abrasive.
Mixing	Stir the material well in the original containers. Add component (B) to component (A). Mix with an electric hand mixer for about 3 minutes until all the coloured streaks have disappeared in the mix, at the sides and on the bottom of the can. Mix at low speed so that as little air as possible is entrained (500 rpm max.). The pot life begins when the resin hardener are mixed. It is shorter at high temperatures and longer at low temperatures. The greater the quantity mixed, the shorter the pot life. To obtain longer workability at high temperatures, the mixed adhesive may be divided into portions. Another method is to chill components (A) and (B) before mixing them.
Application	The well-mixed Sikadur® -30 is applied with a spatula, trowel or float to both the concrete or timber surface and the steel or Sika® CFRP laminate surface, so that the thickness is 0.5 - 2 mm. The adhesive must be applied with great care to the concrete surface to ensure that all voids are filled and no cavities are left. Reinforcements must be positioned within the open time of the adhesive. This is done with specially prepared supports. It is very important that the steel parts are pressed evenly on to the concrete surface until the adhesive is forced out the sides. Because curing is very rapid at normal temperatures, the supports can be removed after 2 or 3 days (see curing times at different temperatures). As a final check, the loads are tapped with a hammer for cavities. So that the adhesive material can be checked for curing rate and final strength, it is recommended that samples be taken at site and their compressive and/or adhesive strength measured.
Cleaning	Clean tools immediately with Colma Cleaner. Wash hands and skin thoroughly in warm soap water. When uncured, Sikadur® -30 components (A + B), are water-pollutants and should not be discharged into drains, waterways or the ground. Colma Cleaner and Sikadur® -30 residues must always be disposed of in accordance with the regulations. Cured material can only be removed mechanically.
Pot Life (to F.I.P.)*	Min. 30 minutes (at +35°C).

Safety Instructions

Ecology	In a liquid state material contaminates water. Do not dispose of into water or soil but according to local regulations.
Transport	Comp. (A): Non-hazardous. Comp. (B): 8/65 c).
Safety precautions	Product can cause skin irritation. Wear protective clothing (gloves, safety glasses). Cover hands with barrier cream before application. In contact with eyes or membranes, rinse thoroughly with clean warm water immediately and seek medical attention without delay.
Toxicity	Com p. (A): Class 4, under the relevant health and Safety codes. Observe warning on packing. Comp. (B): Non-Toxic.

A.7 **Table 1 Concrete mix design form**

Stage	Item		Reference or calculation	Values
1	1.1	Characteristic strength	Specified	30 N/mm² at 28 days; Proportion defective 5% per cent
	1.2	Standard deviation	Fig 3	____ N/mm² or no data 8 N/mm²
	1.3	Margin	C1	(k = 1.64 × 8 = 13.12 N/mm²
	1.4	Target mean strength	C2	13.12 + 30 = 43.1 N/mm²
	1.5	Cement type	Specified	OPC/SRPC/RHPC
	1.6	Aggregate type: coarse Aggregate type: fine		crushed un crushed
	1.7	Free-water/cement ratio	Table 2, Fig 4	0.55 — Use the lower value
	1.8	*Maximum free-water/cement ratio*	*Specified*	____
2	2.1	Slump or V-B	Specified	Slump 30-60 mm or V-B ____ s
	2.2	Maximum aggregate size	Specified	10 mm
	2.3	Free-water content	Table 3	230 kg/m³
3	3.1	Cement content	C3	230 ÷ 0.55 = 420 kg/m³
	3.2	*Maximum cement content*	*Specified*	____ kg/m³
	3.3	*Minimum cement content*	*Specified*	____ kg/m³ — Use if greater than Item 3.1 and calculate Item 3.4
	3.4	Modified free-water/cement ratio		____
4	4.1	Relative density of aggregate (SSD)		2.7 known/assumed
	4.2	Concrete density	Fig 5	2370 kg/m³
	4.3	Total aggregate content	C4	2370 − 230 − 420 = 1720 kg/m³
5	5.1	Grading of fine aggregate	BS 882	Zone 3
	5.2	Proportion of fine aggregate	Fig 6	42.5% per cent
	5.3	Fine aggregate content	C5	1720 × 0.425 = 731 kg/m³
	5.4	Coarse aggregate content	C5	1720 − 731 = 989 kg/m³

Quantities	Cement (kg)	Water (kg or l)	Fine aggregate (kg)	Coarse aggregate (kg)
per m³ (to nearest 5 kg)	420	230	730	990
per trial mix of 0.3 m³	33.8	18.54	58.83	79.79

Items in italics are optional limiting values that may be specified (see Section 7).

1 N/mm² = 1 MN/m² = 1 MPa (see footnote on page 8).

OPC = ordinary Portland cement; SRPC = sulphate-resisting Portland cement; RHPC = rapid-hardening Portland cement.

Table 2 Approximate compressive strengths (N/mm²) of concrete mixes made with a free-water/cement ratio of 0.5

Type of cement	Type of coarse aggregate	Compressive strengths (N/mm²) Age (days) 3	7	28	91
Ordinary Portland (OPC) or sulphate-resisting Portland (SRPC)	Uncrushed	22	30	42	49
	Crushed	27	36	49	56
Rapid-hardening Portland (RHPC)	Uncrushed	29	37	48	54
	Crushed	34	43	55	61

1 N/mm² = 1 MN/m² = 1 MPa (see footnote on earlier page).

Table 3 Approximate free-water contents (kg/m³) required to give various levels of workability

Slump (mm)		0-10	10-30	30-60	60-180
Vebe time(s)		>12	6-12	3-6	0-3
Maximum size aggregate (mm)	**Type of aggregate**				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

Note: When coarse and fine aggregates of different types are used, the free-water content is estimated by the expression

$$\tfrac{2}{3} W_f + \tfrac{1}{3} W_c$$

where W_f = free-water content appropriate to type of fine aggregate

and W_c = free-water content appropriate to type of coarse aggregate.

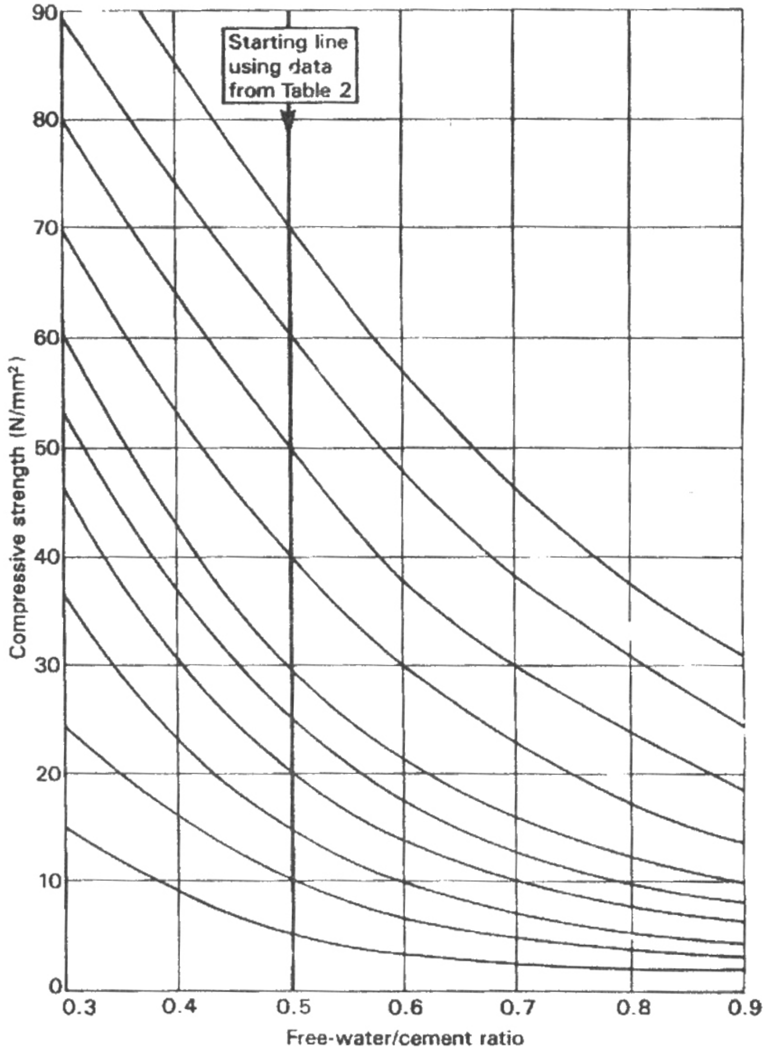


Figure 4 Relationship between compressive strength and free-water/cement ratio

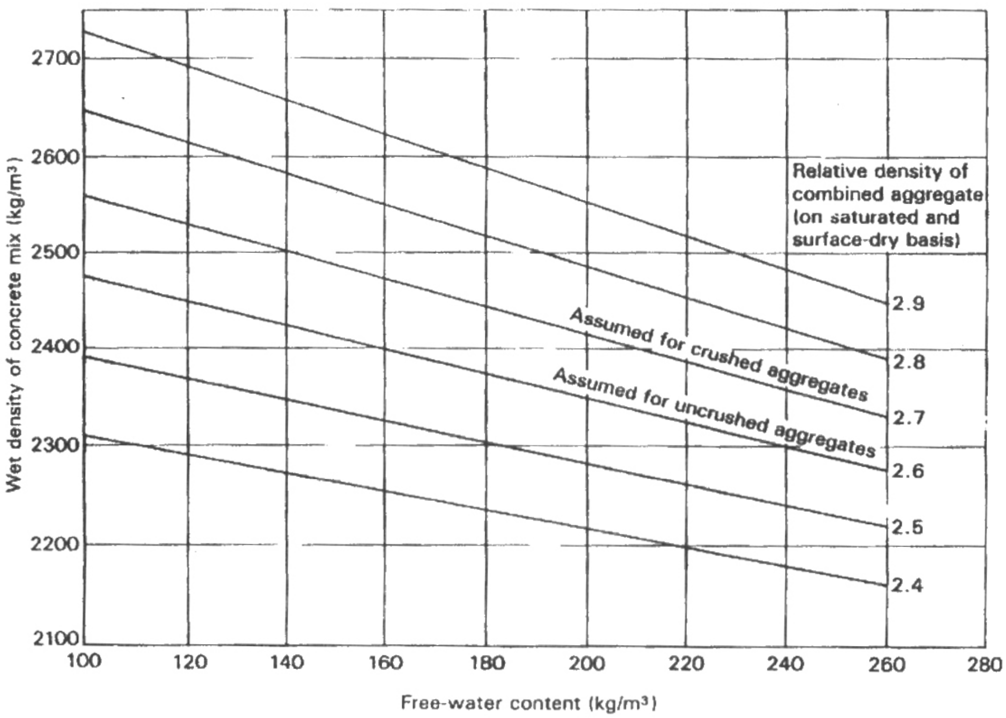


Figure 5 Estimated wet density of fully compacted concrete

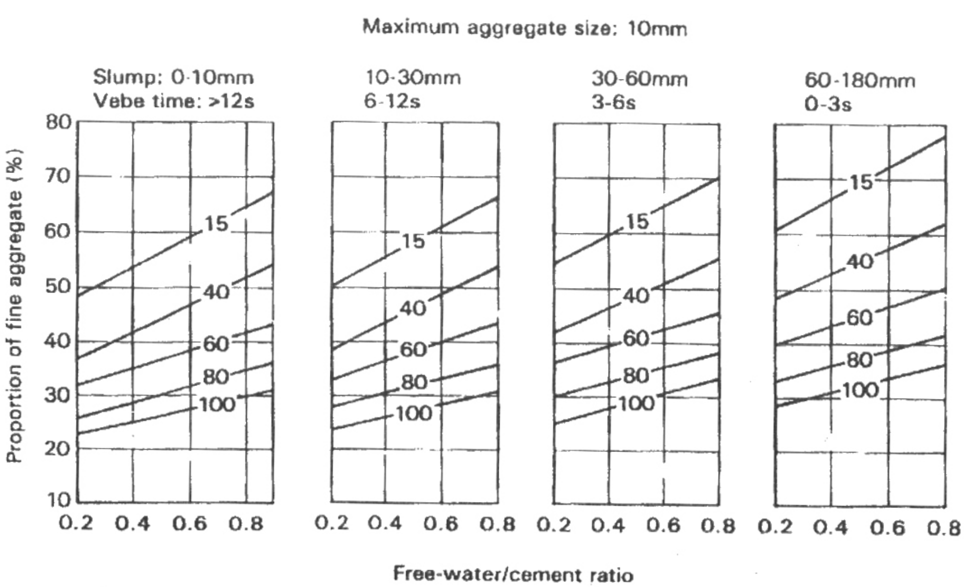


Figure 6 Recommended proportions of fine aggregate according to percentage passing a 600 μm sieve

A.8 Compressive Concrete Strength Tests

Series NO.	Date of cast	Date of test	Age (days)	Cubes dimension (mm)	Weight (kg)	Failure load (kN)	Strength (N/mm^2)	Mixture slump (mm)
A	1.6.2009	4.6.2009	3	100*100*100	2.585	245	24.5	60
					2.54	260	26.0	
					2.51	275	27.5	
		8.6.2009	7	100*100*100	2.58	330	33.0	
					2.58	360	36.0	
					2.55	355	35.5	
		17.8.2009	77	100*100*100	2.58	490	49.0	
					2.54	500	50.0	
					2.34	530	53.0	
	15.6.2009	18.6.2009	3	100*100*100	2.55	275	27.5	60
					2.54	265	26.5	
					2.57	265	26.5	
		22.6.2009	7	100*100*100	2.58	360	36.0	
					2.4	355	35.5	
					2.35	310	31.0	
		13.9.2009	88	100*100*100	2.54	455	45.5	
					2.55	385	38.5	
					2.6	480	48.0	

A.9.1 Load Deflection Results - slab A1

Load (kN)	Deflection (mm)	Remarks
0	0	
30	1.75	
60	3.75	
90	4.65	
120	5.5	
150	6.35	Flexural cracks
180	8.35	
210	9	
240	9.5	
250	10.8	Shear cracks
260	11.9	
270	12.15	
280	12.4	
300	12.87	
310	13.05	
320	13.25	
330	13.3	
340	17.65	

A.9.2 Load Deflection Results - slab A2

Load (kN)	Deflection (mm)	Remarks
0	0	
30	1.03	
60	2.83	
90	4.53	
120	5.13	
150	5.58	
180	6.03	Flexural cracks
210	7.03	
240	7.43	
270	8.83	Shear cracks
300	9.43	
330	9.83	
340	14.93	

A.9.3 Load Deflection Results - slab A3

Load (kN)	Deflection (mm)	Remarks
0	0	
30	0.73	
60	1.63	
90	2.53	
120	4.03	
150	4.53	
180	5.43	
210	6.23	
240	6.9	
270	7.9	
300	8.5	Shear cracks
330	9	
340	12.5	

A.9.14 Load Deflection Results - slab B1

Load (kN)	Deflection (mm)	Remarks
0	0	
30	0.8	
60	1.4	
90	3	
120	3.5	
150	5	Flexural cracks
180	5.45	
210	6.9	
240	7.35	
270	8.8	Shear cracks
300	9.7	
330	10.75	
340	15.9	

A.9.5 Load Deflection Results - slab B2

Load (kN)	Deflection (mm)	Remarks
0	0	
30	0.65	
60	1.45	
90	2.03	
120	2.52	
150	3.48	
180	3.93	
210	4.35	Flexural cracks
240	4.77	
270	5.21	
300	5.65	Shear cracks
330	6.23	
340	10.23	

A.9.6 Load Deflection Results - slab B3

Load (kN)	Deflection (mm)	Remarks
0	0	
30	0.59	
60	1.09	
90	1.89	
120	2.04	
150	3.39	
180	3.89	
210	4.09	
240	4.69	
270	4.89	
300	5.39	
310	5.89	
320	6.09	
330	6.49	
340	7.09	Shear cracks